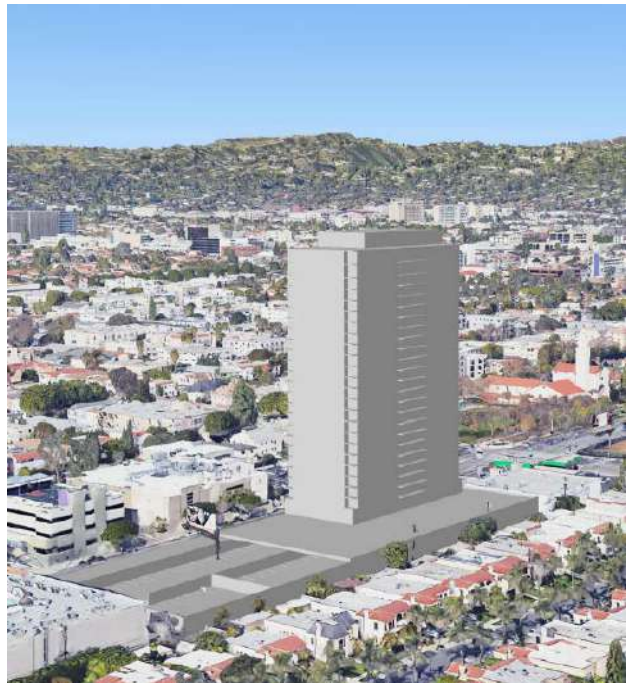


Report

GEOTECHNICAL INVESTIGATION REPORT PROPOSED MIXED-USE DEVELOPMENT 1056 LA CIENEGA BOULEVARD PROJECT 1022 - 1066 LA CIENEGA BOULEVARD, LOS ANGELES, CA



Prepared for

1050 La Cienega, LLC

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March 30, 2022



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March 30, 2022
Project No.: 21086A

Mr. Eric Snow
1000 Sansome Street. 1st Floor
San Francisco, CA 94111

**Subject: Geotechnical Investigation Report
 Proposed Mixed-Use Development
 1022-1066 La Cienega Boulevard
 Los Angeles, California 90035**

Dear Mr. Snow:

This report presents the results of the geotechnical investigation by GeoPentech, Inc. (GeoPentech) for the proposed mixed-use development to be located at 1022-1056 La Cienega Boulevard in Los Angeles, California (also referred to as 1056 La Cienega). This investigation was performed in general accordance with our agreement dated September 7, 2021, GeoPentech proposals dated August 4, 2021, September 29, 2021, and January 28, 2022, and the authorizations dated August 20, 2021, September 29, 2021, and January 28, 2022.

This report provides geotechnical recommendations for the design and construction of the project in accordance with the plans provided to us. Prior and current field and laboratory test results, as well as geologic hazard evaluation and details of the ground motion evaluation, are also included in the report.

Thank you for providing GeoPentech with the opportunity to participate in this project. If you have any questions or require additional information, please call.

Very truly yours,

GeoPentech, Inc.

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1.0 INTRODUCTION

This report presents the results of GeoPentech’s geotechnical investigation for the proposed mixed-use development that includes a high-rise tower (Tower) and the associated low-rise Podium (Podium) to be located at several parcels with addresses 1022 through 1066 La Cienega Boulevard in Los Angeles, California (34.057649° N, -118.375808° W, referred to as 1056 La Cienega). The location of the project site is shown on Figure 1. The site includes the following parcels:

Street Number (S. La Cienega Blvd)	APN
1022	5087-001-040
1024, 1028	5087-001-041
1034	5087-001-023
1036, 1038, 1044, 1048, 1054	5087-001-024
1056, 1060, 1066	5087-001-042

2.0 PROJECT DESCRIPTION

Our understanding of the project is based on architectural drawings provided by SCB Architects dated June 15, 2021, and structural input from Mr. Tony Ghodsi of Englekirk (Structural Engineer).

We understand that the proposed development includes the design and construction of 27 story high-rise tower on the northern part of the site with includes three parking levels (one subterranean and two above ground) covering most of the site and an amenity deck covering norther part of the site (collectively referred to as podium). The height of the tower will be about 290 feet above grade and the finish floor of the subterranean parking level will extend about 10 feet below grade. The approximate extents of the tower and podium portions of the proposed structure are shown on Figure 2.

We understand that the design for this structure is being carried out in conformance with the 2019 California Building Code (CBC 2019) and ASCE 7-16 requirements using the performance-based design procedure specified by the Los Angeles Tall Buildings Structural Design Council (LATBSDC, 2020). Generalized preliminary loading conditions were provided to us by the Structural indicating an average bearing pressure of about 7,000 psf under the tower and an average bearing pressure of 1,100 psf under the podium portion of the project.

This report presents the results of GeoPentech’s geotechnical investigation as well as design recommendations for the Tower and the associated Podium portion of the proposed mixed-use development.

3.0 SCOPE OF WORK

GeoPentech’s scope of work for this report included the following:

- Review of Existing Information – GeoPentech reviewed existing geotechnical, geologic, and seismic information for the site as well as the currently proposed development plans.
- Field Investigation and Laboratory Testing – Drilled four (4) borings, to depths between 50 and 200 feet below the existing ground surface, and performed three (3) CPTs, to depths between 66 and 91.5 feet below the existing ground surface at the site to further investigate the nature and stratigraphy of the subsurface materials below the currently proposed development, and to obtain soil samples for laboratory testing. The borings were drilled using rotary-wash and hollow-stem auger drilling equipment. Performed laboratory testing on selected samples including moisture and density, wash analysis, Atterberg limits, consolidation, and direct shear tests.
- Geologic-Seismic Hazards Evaluation – Evaluated site subsurface conditions, geologic setting, and assessed seismic conditions and geologic-seismic hazards and their potential impact on the subject project.
- Ground Motion Evaluation – Completed a site-specific ground motion hazard analysis in accordance with the requirements of the 2019 CBC and ASCE 7-16.
- Engineering Analysis – Performed engineering evaluation of the geotechnical data to develop recommendations for the design of foundations, walls below grade, shoring, excavation, earthwork criteria, and paving.
- Preparation of this report.

4.0 EXISTING SITE CONDITIONS

The site is currently unoccupied and consists of a rough graded site with occasional shrubs and no surface improvements such as pavements, landscaping, or hardscapes. The site is relatively flat with a surface area of approximately 78,000 square feet. As shown in Figure 2, the site is bounded by La Cienega Boulevard to the west, an existing single-story commercial building at 1018 La Cienega Boulevard to the north (located approximately 230 ft south of West Olympic Boulevard), an existing

3-story commercial building to the south (located approximately 110 ft north of Whitworth Drive), and adjacent to existing residential homes to the east.

5.0 FIELD EXPLORATION AND LABORATORY TESTING

5.1 Available Subsurface Data

Subsurface data of the subject site consists of previous explorations by Calwest Geotechnical (2008), AGI Geotechnical (2009), and Geotechnologies (2012). The following presents a summary of each investigation:

Calwest Geotechnical (2008): drilled five (5) borings using either screw auger or hollow stem auger methods to depths between 25 and 50 feet and groundwater was encountered at depths between 17 and 17.5 feet below the ground surface. The locations of these borings were not available. As such, while data has been considered, it is not shown in our plan and sections in this report. The logs of the borings from this investigation are included in Appendix A.

AGI Geotechnical (2009): performed two (2) hollow stem auger borings (B-1 and B-2) to depths of 51.5 and 71.5 feet, as shown on Figure 2. Groundwater was encountered at depths of 17 and 16.5 feet below ground surface in borings B-1 and B-2, respectively. Logs of the borings by AGI are presented in Appendix A.

Geotechnologies (2012): drilled two (2) mud-rotary borings (B1 and B2) to depths of 40 and 70 feet, as shown on Figure 2. Groundwater was encountered at depths of 18 and 17 feet below ground surface in borings B1 and B2, respectively. Select samples from the two borings were tested in the laboratory. Logs of the prior borings by Geotechnologies are presented in Appendix A along with the associated laboratory test results from this investigation.

5.2 Borings

Four (4) borings (GP-1, GP-2, GP-3, and GP-4) were completed by GeoPentech at the site to supplement the data collected from previous subsurface investigations. Logs of the borings are presented in Appendices B. GP-1, GP-2, GP-3, and GP-4 were advanced to depths of 100.5, 50.5, 200, and 176.5 feet below ground surface, respectively, at the locations shown in Figure 2. GP-1, GP-2, and GP-3 were drilled using 4 $\frac{3}{8}$ -inch-diameter rotary wash-type drilling equipment, and GP-4 was drilled using 8-inch diameter hollow-stem auger drilling equipment. Standard Penetration Test (SPT) samples, modified California samples, and pitcher barrel samples were collected during drilling. The drilling was subcontracted to BC2 Environmental, who provided all drilling equipment, crew, and supplies. The work was performed under the supervision of a registered civil engineer who monitored

the drilling operations and prepared a field record of soils observed and drilling conditions. Details of the current explorations and the logs of the borings are presented in Appendix B.

5.3 Cone Penetration Testing

Three (3) CPTs (CPT-1, CPT-2, and CPT-3) were performed by GeoPentech at the locations shown in Figure 2. CPT-1, CPT-2, and CPT-3 were advanced to refusal depths of about 91.5, 74, and 66 feet below the existing ground surface, respectively. The locations of all CPTs are shown in Figure 2.

The CPT work was subcontracted to ConeTec, Inc. who provided all CPT equipment, crew, and supplies. Details of the CPTs are presented in Appendix C. The work was performed under the supervision of a registered civil engineer who monitored the testing.

5.4 Laboratory Tests

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to evaluate the pertinent engineering properties of the soils. The following tests were performed:

- Moisture content and dry density
- Passing No. 200 sieve (wash) and sieve distribution
- Atterberg Limits
- Corrosion suite
- Direct shear
- Consolidation

The geotechnical testing was conducted at the laboratory facilities of AP Engineering & Testing, Inc. in Pomona, California. The tests were performed in general accordance with applicable procedures of the American Society for Testing and Materials (ASTM). The complete results of laboratory tests along with the test results are presented in Appendix D. The type of test or the test result is also summarized on the boring logs in Appendix B.

5.5 Surface Wave Geophysical Survey

Geophysical surveys were performed to measure shear-wave (S-wave) velocity within the soil strata underlying the project site. The geophysical investigation consisted of surface wave surveys using Multichannel Analysis of Surface Waves (MASW) and Refraction Microtremor (ReMi) methods. The geophysical measurements were performed along three survey lines (SW21-1 through SW21-3) on August 26, 2021. The locations of all survey lines are shown on Figure 2.

The geophysical data were collected and processed under the supervision of a California-licensed Professional Geophysicist. Details and results of the geophysical survey can be found in Appendix E.

6.0 GEOLOGIC AND SEISMIC CONDITIONS

6.1 Regional Geology and Seismicity

Regionally, the site is located near the boundary of the Peninsular Ranges physiographic province and the Transverse Ranges physiographic province. Northwest trending mountains and faults characterize the Peninsular Ranges, while east-west trending mountains and faults characterize the Transverse Ranges. Locally, the site is within the Los Angeles Basin, about 2½ miles south of the Santa Monica Mountains range front and about 2½ miles north of Baldwin Hills at the northern edge of the Los Angeles Basin, as depicted in Figure 3a. The Baldwin Hills are an expression of the Newport-Inglewood structural zone, which is comprised of a complex system of faults and folds. The Baldwin Hills are composed primarily of Quaternary age sedimentary rocks, locally overlain by Holocene age alluvium in the intervening drainages and lowlands (Figures 3a and 3b; CGS, 2012).

As shown on the geologic map in Figure 3a, the site is located approximately 1½ miles north of Ballona Creek, within Young Alluvial Valley Deposits (“Qya” as shown on the geologic legend in Figure 3b). The Young Alluvial Valley Deposits are regionally described as Holocene to Late Pleistocene age, unconsolidated to slightly consolidated clays, silts, sands, and gravels deposited along stream valleys and alluvial flats of larger rivers (Figures 3a and 3b; CGS, 2012).

The site is located within a seismically active region of southern California, as indicated on Figure 4a. Recent examples of the seismic activity in the region include the M6 1987 Whittier Narrows earthquake and the M6.7 1994 Northridge earthquake. Figure 4a shows the site location relative to mapped active faults in the region, as identified by the US Geological Survey (USGS, 2021). The site is not crossed by any known active faults with late Quaternary surface displacement. Significant faults near the site mapped with late Quaternary surface displacement include the Newport-Inglewood Fault (located less than 3 km to the southwest); the Santa Monica Fault (located about 4 km to the northwest); and the Hollywood Fault (located about 4 km north). The San Andreas Fault is located approximately 60 km to the northeast.

Potentially active blind thrust faults are also believed to exist in the region, as shown in Figure 4b. These blind thrust faults are not expressed at the surface but are inferred to exist based on indirect information, such as seismicity and folded stratigraphy. Recognition of the existence of blind thrust faults in the region was largely triggered by the occurrence of the 1987 Whittier Narrows earthquake. As shown on Figure 4b, the site is located on the hanging wall of the Compton blind thrust and on the footwalls of the Puente Hills (LA), Puente Hills, and Elysian Park blind thrust faults.

6.2 Site Geology and Subsurface Conditions

The site is underlain by Holocene- to late Pleistocene-age, Young Alluvial Valley Deposits (referred to herein as Quaternary alluvium), as shown on the geologic map in Figure 3a. This is consistent with the materials logged by others during prior investigations as well as GeoPentech's findings during our recent site investigation. Specifically, at the site, the Quaternary alluvium was overlain by a shallow artificial fill cover.

Generalized cross-sections A-A' and B-B' showing the subsurface profiles are shown in Figures 5a and 5b, and the locations of the geologic cross-sections are shown in Figure 2. Descriptions of each of the geologic units are discussed below.

Artificial Fill

The current and previous borings advanced at the site encountered artificial fill ranging from approximately 2½ to 7½ feet thick. The fill generally consisted of dark brown silty sand to sandy silt (SM to ML) and sandy clay (CL). This fill was generally characterized as medium-dense (for sandy material) or stiff (for silty and clayey material), and moist. The fill is likely the result of past demolition and construction activities at the site. Note that deeper fill, including debris, which was not encountered in the borings, may also exist on site, and the density/strength of the fill may also vary across the site.

Quaternary Alluvium

The artificial fill at the site is underlain by Quaternary age alluvium. Within the upper 55-60 feet of the subsurface, the alluvium encountered was generally fine-grained consisting of mostly clays (CL and CH) and silts (ML) with some coarse-grained zones consisting mostly of sands with various amounts of fines (SM, SP, and SW). Below the upper fine-grained soils, the alluvium encountered was mostly coarse-grain consisting of silty sand (SM, SP, SP-SM) with occasional layers of clay (CH, and CL), and silt (ML) to about 95 to 110 ft below ground. Below this zone, another predominantly fine-grained alluvium zone was encountered consisting of mostly lean clays (CL) to about 145 to 155 ft below ground. The soils below this zone to the maximum depth explored of 200 feet were generally coarse-grained consisting of sands with various amounts of fines (SP-SM). Overall, the alluvium was medium-dense to very-dense (for coarse-grained soils) or medium-stiff to very hard (for fine-grained), generally becoming more stiff/dense with greater depth.

6.3 Groundwater

Perched groundwater was observed during drilling in the current investigation at a depth of 8 feet below ground surface in GP-1. Groundwater was also encountered in boring GP-4 at a depth of 20 feet below ground surface. Previous investigations by Geotechnologies and AGI Geotechnical

observed groundwater at depths ranging from 15 to 18 feet below the existing ground surface. Based on a review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (CGS, 1998), the historic high groundwater level beneath the site is estimated to be about 15 feet below the ground surface.

It should be recognized that groundwater levels can fluctuate over time, depending on seasonal rainfall and other influences (i.e., irrigation). Furthermore, there may be a potential for perched water to occur locally in sandy zones of the alluvial deposits above the static groundwater level. In addition, recent changes in policies for the use of stormwater infiltration could result in changing seepage conditions at shallow depths across the region.

7.0 POTENTIAL GEOLOGIC AND SEISMIC HAZARDS

An evaluation of the potential geologic hazards is presented in the following sections.

7.1 Surface Fault Rupture

The site is not located within a currently established Alquist-Priolo (AP) Zone based on a review of the Earthquake Zones of Required Investigation for the Beverly Hills Quadrangle (CGS, 2018). Additionally, the site is not located within 1,000 feet of a mapped Holocene-active fault based on a review of mapping by (USGS, 2021), as shown on Figure 4a. The site is located as close as about 1,800 feet east of the Earthquake Fault Zone for the Newport-Inglewood Fault. Therefore, the site is not considered susceptible to surface fault rupture hazards.

7.2 Seismic Shaking

We understand that the design for the project is being carried out in conformance with the 2019 CBC and ASCE 7-16 requirements using the performance-based design procedure specified by LATBDC, 2020. A site-specific hazard evaluation that included both Probabilistic Seismic Hazard Analysis (PSHA) and Deterministic Seismic Hazard Analysis (DSHA) has been carried out for the site. This analysis and its detailed results are presented in Appendix F of the report. To fulfill the seismic design requirements, the following site-specific response spectra are developed:

- A “Maximum Considered Event” uniform hazard spectrum with risk-targeted, maximum rotated ordinates at 5% damping; also known as a site-specific MCE_R response spectrum (corresponding to a 1% probability of collapse in a 50-year period; i.e., a modified 2,475-year return period spectrum). Note that due to the proximity of the Newport-Inglewood Fault, two-component MCE_R -level spectra are provided: a Fault Normal (FN) MCE_R spectrum and a Fault Parallel (FP) MCE_R spectrum.

- A “Service-Level Earthquake” uniform hazard spectrum with average horizontal spectral ordinates at 2.5% damping (corresponding to a 50% probability of exceedance in a 30-year period; i.e., a 43-year return period)

For completeness, the code-compliant, site-specific “Design Level” or DRS uniform hazard spectrum with risk-targeted, maximum-rotated ordinates at 5% damping has also been provided.

7.3 Liquefaction Potential

Liquefaction potential is greatest where the groundwater level is shallow and submerged loose to medium-dense sand occur within a depth of about 50 feet or less below the ground surface. Liquefaction potential generally decreases as fines and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

According to the CGS map of Earthquake Zones of Required Investigation for the Beverly Hills Quadrangle (CGS, 2018), and the County of Los Angeles Seismic Safety Element (1990), the site is located within an area identified as having a potential for liquefaction.

Based on our review of the available previous information and incorporating the recent exploration data, the liquefaction potential in localized zones of the alluvial soils mostly in the upper 50 to 60 ft of the site below the historic high groundwater during a design earthquake exists. While most of the soils in the upper 50 to 60 feet of the soil profile are predominantly clayey, there are some interbedded sandy layers of a relatively limited extent that could present a residual hazard of due to liquefaction. Based on our calculations, we estimate that liquefaction-induced settlements of these layers could be up to about 1 to 2 inches under the design-level shaking.

As indicated later in this report, our recommendation for the building foundation system would mitigate this hazard. Further details are included in Section 8 of this report.

7.4 Seismically-Induced Dry Sand Settlement

Seismically-induced settlement may also be caused by unsaturated loose to medium-dense granular soils densifying during ground shaking. Uniform settlement beneath a given structure would cause minimal damage; however, because of variations in distribution, density, and confining conditions of the soils, seismically-induced settlement is generally non-uniform and can cause serious structural damage.

As part of the site development, the upper 15 to 20 feet of the site will be excavated and the soils removed for the new basement level which will extend to approximately the groundwater, thereby removing all the dry soils potentially susceptible to this type of seismic settlement. Accordingly, the potential for seismically-induced dry sand settlement at the site for this project configuration is considered to be negligible.

7.5 Subsidence

Ground surface subsidence generally results from the extraction of fluids or gas from the subsurface that can result in the gradual lowering of the overlying ground surface. Based on the available information from the California Department of Water Resources (2014) the site is located within a groundwater basin with documented historic subsidence and is designated as having low to medium estimated potential for future land subsidence. Therefore, the potential for future subsidence at the site is considered low to medium. Note that the potential impacts from subsidence would likely be distributed regionally and would not result in impacts at the site significantly different from those experienced across the region.

7.6 Flooding

According to FEMA (2008), the site is not located within a defined floodplain or floodway boundary. The site has been assigned a FEMA Flood Zone X, which indicates “areas determined to be outside the 0.2% annual chance floodplain”. As such, flooding is not considered a hazard at the site.

7.7 Seiches and Inundation (Water Storage Facilities)

This potential hazard is associated with seiches (water waves created when a body of water is shaken that have the potential to overtop a water storage facility) and inundation due to water storage facility failure. No major water-retaining structures are located immediately up gradient of the site; however, the site is located within a mapped inundation area for the Lower Franklin Reservoir according to the Safety Element of the Los Angeles City General Plan (1996) and Department of Water Resources (DWR). As such, there is a potential for inundation at the site. Note that DWR regulates water storage facilities for public safety and additional information on specific facilities is available through their website.

7.8 Tsunami

A tsunami is a sea wave generated by a large submarine landslide or an earthquake-related ground deformation beneath the ocean. Historic tsunamis have been observed to produce a run-up on shore of several tens of feet in extreme cases. The site is located at an elevation of about 90 to 100 feet above mean sea level and is relatively far from the shoreline. Additionally, the site is not located within a tsunami inundation area as designated by the Safety Element of the Los Angeles City General Plan (1996). As such, the site is not considered susceptible to tsunami hazards.

7.9 Landslide

A potential for landsliding is often indicated in areas of moderate to steep terrain that are underlain by unfavorably oriented geologic discontinuities. The site is located on relatively level terrain and no landslides are mapped in the vicinity of the site (CGS, 1998). In addition, the site is not in a designated



earthquake-induced landslide hazard zone (CGS, 2018). Therefore, the potential for landsliding is considered negligible.

7.10 Volcanic Eruption

Potential hazards from volcanic eruptions include both lava flows and ash falls from relatively nearby volcanoes. No active volcanic sources are present in the Los Angeles basin. Therefore, the potential for damage at the site due to volcanic eruption is considered to be negligible.

7.11 Erosion

The majority of the ground surface at the site is relatively level and is or will be covered with asphalt or concrete pavements. As such, erosion is not considered a hazard at the site.

7.12 Methane Gas

Based on a review of the Methane and Methane Buffer Zones map (2004) prepared by Los Angeles Department of Building & Safety (LADBS), the site is located within the boundaries of a methane zone, as defined by the City of Los Angeles. We recommend that prior to design and construction, a methane specialist be consulted and a site-specific study be conducted to evaluate the potential impact of methane.

Based on a review of the California Division of Oil, Gas, and Geothermal Resources (DOGGR) Well Finder online GIS data system, the site is located within the Beverly Hills oil field and no wells are shown at the site. One well by Chevron, "Saturn Corehole #1" is indicated about 650 ft south of the site, but it is indicated as plugged and abandoned. However, note that the records provided by oil well drilling companies are voluntary and not all wells may be shown on available maps.

8.0 GEOTECHNICAL RECOMMENDATIONS

Based on our understanding of the project and the results of our investigation, the proposed development is feasible from a geotechnical point of view. Key geotechnical considerations are discussed below:

Compressible soils: One of the key geotechnical considerations for the project will be the impact of potentially compressible soils that were encountered during our investigation. Medium stiff to stiff clays and silts encountered are considered compressible and subject to settlement under heavy loads expected in particular, below the proposed tower (average bearing pressure of 7,000 psf), which could result in significant consolidation settlement if not mitigated.

Potentially liquefiable soils: Zones of medium-dense sands and medium-stiff silts generally in the upper 60 feet below ground were encountered that could be subject to liquefaction under strong ground shaking. The thickness of the potentially liquefiable layers varies across the site, and generally,

the materials are of a limited extent and are interbedded. However, in aggregate, these materials do present a potential impact due to the additional settlement that could occur which needs to be addressed for the design.

8.1 Seismic Design Parameters

In developing the preliminary seismic design parameters in accordance with the 2019 CBC and ASCE 7-16 Standard, a seismic site class D was selected based on a review of the shear-wave velocity data recently collected at the site (see Appendix F). $S_S = 2.069g$ and $S_1 = 0.738g$ are the mapped seismic values provided by USGS. Using ASCE 7-16, Section 21.4, the site-specific seismic response spectra and design parameters for new structures at the project site are developed in Appendix F and key design parameters are summarized below.

$S_{DS} = 1.563 g$, based on 90% of the spectral acceleration at a period of 0.3-seconds

$S_{D1} = 1.093 g$, based on the spectral acceleration at a period of 1.0-second

$S_{MS} = 2.345 g$, based on 1.5 times S_{DS}

$S_{M1} = 1.640 g$, based on 1.5 times S_{D1}

Further details of the development of the seismic hazard analysis and the site-specific design response spectra for the project are included in Appendix F.

8.2 Foundation Recommendations

Loading conditions provided to us by the Structural Engineer indicate an average bearing pressure of about 7,000 pounds per square foot (psf) under the footprint of the mat foundation supporting the tower, and an average bearing pressure of 1,100 psf under the footprint of the podium. Considering one level below grade and accounting roughly for the thickness of the foundations, excavation and removal of the upper 15 to 20 feet of soils at the site are anticipated.

Under these conditions, without mitigation, our calculations (see Appendix G) indicate up to about 11 to 13 inches of static settlement could occur under the tower with about 1/2 to 1 inch under the Podium. In addition, seismically-induced settlements could be an additional 1 to 2 inches. Given the magnitude of these estimates, mitigation measures would be required to control total settlements (static and seismic) under the tower and address seismic settlements under the podium.

Several options in supporting the tower and podium structures have been considered including mat foundations on the native or improved ground, or deep foundations. Ground improvement techniques including stone columns and deep soil mixing (DSM) were also considered. Given the expected magnitude of settlements, a deep foundation system or mat foundation supported on improved ground is recommended to control settlements under the tower. Under the podium,



settlements are expected to be lower and a mat foundation on the native or improved ground as well as deep foundations could be used to control settlements. If two different foundation systems are considered under the podium and tower, a structural joint will be required to separate the two structures.

Based on discussion with the project team, we understand the following foundation systems are currently being considered for the project:

- Drilled pile foundations under the tower and a continuous mat foundation under the podium portion of the project, with a structural joint between tower and podium. No ground improvement would be required for this approach.
- Mat foundation supported on improved ground with Deep Soil Mixing (DSM) under the tower and a separate mat foundation on the native (unimproved) ground under the podium portion of the project, with a structural joint between tower and podium.
- Mat foundation on improved ground with Deep Soil Mixing (DSM) under the entire project without any structural joints.

These options are considered feasible from a geotechnical point of view. Further recommendations for ground improvement, mat foundations, and drilled piles are discussed below.

Ground Improvement

Based on our conversation with ground improvement designers, supporting both tower and podium on a mat foundation bearing on improved ground with DSM is a technically feasible option for the project. The design and construction of DSM should be completed by a separate design-build ground improvement contractor based on the specifics of the technique utilized. The final DSM layout and design will depend on the strategy adopted by the selected ground improvement contractor. Once the selection has been made, the design (including the bearing and lateral capacity as well as modulus of subgrade reaction for a mat foundation supported on the improved ground) should be submitted to us for review so that we can evaluate the proposed plans for consistency with our recommendations and provide feedback to the selected design-build contractor.

It should be noted that in our experience, review and approval of DSM for support of the mat foundation could be subject to a special approval process by Los Angeles Department of Building Safety (LADBS) and could take longer compared to other more common foundation systems such as mat foundations or drilled piles.

Mat Foundation

The podium portion of the project could be supported on a mat foundation resting on native ground. A mat foundation founded on native alluvium at a depth of 15 to 20 ft from existing grade may be

designed using an allowable bearing capacity of 1,500 psf. This value is for dead plus live loads and may be increased by one-third to accommodate transient loads that include wind or seismic loads. We estimate the settlement of the proposed tower supported on a mat foundation in the manner recommended, will be less than 1 inch for an average mat bearing pressure of 1,500 psf. Additional liquefaction settlement on the order of 1 to 2 inches is also anticipated for unimproved ground during a strong seismic event. Differential settlement is estimated to be about half of the total settlement across the mat in either direction.

For structural analyses of the mat foundation supported on undisturbed natural soils at the planned excavation level, a modulus of subgrade reaction, k , of 200 pounds per cubic inch (pci) may be used. This value is a unit value for use with a 1-foot-square area. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K \left[\frac{B+1}{2B} \right]^2$$

Where:

K = unit subgrade modulus

K_R = reduced subgrade modulus

B = foundation width

Lateral loads may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.35 may be used between the mat foundation and the underlying native soils. The allowable passive resistance of undisturbed natural soils is recommended to be equal to the pressure developed by a fluid with a density of 300 pcf. The allowable passive resistance should be limited to a maximum value of 3,000 psf. The upper foot of the material should be ignored for calculating this value. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and passive resistance of the soils may be combined without reduction in evaluating the total lateral resistance.

The recommended bearing and lateral load design values are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values shall be multiplied by the following factors:



<u>Design Item</u>	<u>Ultimate Design Factor</u>
Bearing Value	3.0
Passive Pressure	2.0
Coefficient of Friction	2.0

Deep Foundations – Drilled Piles

The proposed tower can be supported on deep foundations. A drilled pile foundations system (Cast-in-place drilled pile) with pile tips at or below Elevation -30 ft is recommended. Additional details are provided below:

Axial Capacity

The estimated allowable downward and upward capacities of 36-, 42- and 48-inch-diameter drilled cast-in-place concrete piles extending to or below elevation -30 ft into the alluvium are presented in Appendix H. The capacities are based on skin friction only, and end bearing has not been included. The estimated capacities are for dead plus live loads and may be increased by one-third to accommodate transient loads that include wind or seismic loads. The capacities presented are based on the strength of the soil materials; the compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles. Ultimate capacities may be obtained by multiplying the allowable capacities by a factor of 2.

Note that piles in groups should be spaced at least 3 diameters on centers. If the piles are so spaced, no reduction in the axial capacities need be considered due to group action.

Settlement

Settlements of the proposed Tower and Podium structures if supported on drilled piles would depend on the loads and selected pile lengths. However, for preliminary planning purposes and assuming that the axial allowable downward capacities of the piles are on the order of 1000 kips and extending to at least elevation -30 feet, we estimate settlement of an individual drilled pile in the manner recommended to be up to 1 inch. The differential settlement between adjacent similarly loaded pile caps is anticipated to be ½ the total. Note that these settlement estimates should be updated based on the actual configuration and axial loading when available.

Lateral Capacity

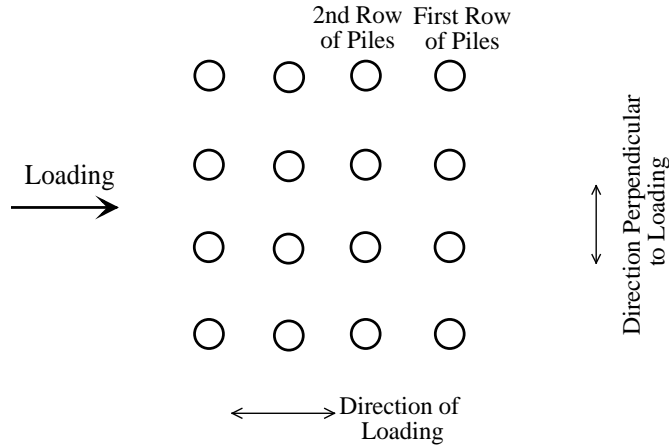


Lateral loads may be resisted by the drilled piles, by the passive resistance of the soils against pile caps and grade beams, and by soil friction between slab and the supporting soils. The lateral resistance of piles, the passive resistance of the soils against pile caps and grade beams, and the frictional resistance may be combined without reduction in evaluating the total lateral resistance.

The lateral capacities of the piles were computed using the computer program LPILE by ENSOFT, Inc. Lateral load capacities and load deflection curves for 36-, 42-, and 48-inch-diameter drilled cast-in-place concrete piles extending into native alluvium under the site are presented in Appendix H. The analyses were performed for top of pile deflection of ¼-inch, ½-inch, and 1-inch and considering piles extending to at least elevation -30 feet. The capacities have been calculated assuming both pinned and fixed-head pile conditions. The capacities presented are based on the strength of the soils; the strength of the pile sections should be checked to verify the structural capacity of the piles.

For piles in groups spaced as shown below and at least 3 pile diameters on centers, no reduction in the lateral capacities need be considered for the first (leading) row of piles in the direction perpendicular to loading. For subsequent rows in the direction of loading, piles in groups spaced closer than 8 pile diameters on centers will have a reduction in lateral capacity due to group effects. Therefore, the lateral capacity of piles in groups, except for the first row of piles, spaced at 3 pile diameters on centers, may be assumed to be reduced by half. The reduction of lateral capacity in the direction of loading for other pile spacings may be interpolated.

A coefficient of friction of 0.35 may be used between the pile cap and the supporting soils. The passive resistance of engineered fill and/or undisturbed natural soils against pile caps and grade beams may be assumed to be equal to the pressure developed by an equivalent fluid with a density of 300 pcf. The upper foot of material should be ignored when calculating this value. A one-third increase in the passive value may be used for wind or seismic loads. The lateral resistance of piles, the frictional resistance, and the passive resistance of the soils against pile caps and grade beams may be combined without reduction in evaluating the total lateral resistance.



Ultimate Design Factors

The recommended values for foundation designs are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values shall be multiplied by the following factors:

<u>Design Item</u>	<u>Ultimate Design Factor</u>
Axial Capacity of Piles	2.0
Lateral Capacity of Piles	1.0
Passive Pressure	1.5
Coefficient of Friction	1.5

In no event, however, shall pile lengths be less than those required for dead-plus-live loads when using the working stress design values.

Pile Installation

Groundwater was encountered during the current and previous field investigation at a depth of 15 to 20 ft, with perch groundwater as shallow as 8 ft below the surface. Therefore, we anticipate that the recommended piles will be installed partially below groundwater. The alluvial soils beneath the site encountered in our investigations consisted of both fine-grained (silty/clayey) and coarse-grained layers. As such, caving potential during pile drilling exists, and provisions to prevent the caving of shaft side walls during construction may be required. Special drilling provisions include but are not limited to, reduced drilling speed and the use of casing and/or drilling fluids. Drilling fluids should be approved prior to use. As some caving and raveling may occur during installation, piles spaced less than five diameters on the center should be drilled and filled alternately, with the concrete permitted

to set at least eight hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight.

Concrete should be pumped from the bottom up through a rigid pipe extending to the bottom of the drilled excavation, with the pipe being slowly withdrawn as the concrete level rises. The discharge end of the pipe should be at least 5 feet below the surface of the concrete at all times during placement. The discharge pipe should be kept full of concrete during the entire placing operation and should not be removed from the concrete until all of the concrete is placed and fresh concrete appears at the top of the pile. The volume of concrete pumped into the hole should be recorded and compared to design volume.

Only competent drilling contractors with experience in the installation of drilled cast-in-place piles in similar soil conditions and depths should be considered for the pile construction. We suggest requesting the piling contractor to submit a list of similar projects along with references for each project.

The drilling of the pile excavations and the placing of the concrete should be observed continuously by a representative of a qualified geotechnical firm to verify that the desired diameter and depth of piles are achieved.

Pile Testing

In conformance with Section 1810.3.3.1.2 of CBC, if the design compressive loads are greater than those determined using the allowable stresses specified in Section 1810.3.2, a pile test program will be required. If testing is required, at least one sacrificial pre-production drilled cast-in-place pile should be installed and tested to confirm the axial capacity of drilled piles. The pre-production pile(s) should include the reinforcing steel. In addition, if needed, the pre-production pile(s) should be tested in accordance with ASTM D1143 to confirm the ultimate axial capacity.

Regardless of the need for the testing as per Section 1810.3.3.1.2 of CBC, integrity testing, such as nuclear logging (gamma-gamma logging), or cross-hole sonic logging, should be performed on two production piles to confirm the absence of anomalies due to construction techniques within the piles (inclusion of soil or voids).

8.3 Uplift and Waterproofing Considerations

As previously discussed, the historic high groundwater level beneath the site is estimated to be at 15 feet bgs. It is assumed that the bottom of the foundations for the proposed tower and the associated

podium will be placed at about 15 to 20 feet below the existing ground surface. For portions of the foundation extending more than 15 feet below existing ground surface, hydrostatic uplift pressure should be incorporated into the design. The uplift pressure can be calculated based on a fluid weight of 62.4 pounds per cubic foot (pcf) and can be resisted by self-weight of the building and foundation.

Note that the foundations, basement walls, and interior slabs should be waterproofed to prevent seepage of water or moisture due to cracks or water migration. Waterproofing should extend at least 5 feet above the design groundwater level (i.e. to 10 feet below existing ground surface) and a qualified waterproofing consultant should be retained for recommendations of suitable waterproofing applications behind all walls below grade, foundations, and slabs if necessary.

8.4 Walls Below Grade

Lateral Earth Pressure

Subterranean parking and basement walls should be designed to resist lateral earth pressures plus any surcharges from adjacent loads. Retaining walls that are free to move and rotate at the top, such as cantilever walls, may be designed for an active pressure imposed by an equivalent fluid weighing 35 pcf. Permanent basement walls that are restrained at the top of the wall should be designed to resist an at-rest lateral earth pressure imposed by an equivalent fluid weighing 50 pcf. The recommended earth pressures are calculated assuming that a drainage system will be installed, so that hydrostatic pressure will not develop behind the subterranean walls.

In addition to the recommended earth pressure, the upper 10 feet of walls below grade and retaining walls adjacent to areas subject to vehicular traffic should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the walls due to normal vehicular traffic. If the traffic is kept back at least 10 feet from the top of walls, the traffic surcharge can be neglected. For the basement walls adjacent to the at-grade structures, surcharge pressures can be provided on a case-by-case basis once the estimated loading conditions from these structures and the details of the foundations are provided to us.

Loads from equipment surcharge imposed on the adjacent ground may be computed using a coefficient of 0.5 times the uniform load applied.

In addition to the above-mentioned lateral earth pressures, the walls below grade should be designed to support a seismic lateral pressure of $22H$ (psf) applied uniformly along the wall height H (in feet). This seismic load is a directly calculated value and can be used as is. When designing for seismic loads, the seismic lateral earth pressure should be combined with the active earth pressure mentioned

previously. If designing for static loading condition only, the at-rest lateral earth pressure should be used.

Drainage

Walls below grade and retaining walls should be designed to resist hydrostatic pressures (equivalent fluid pressure of 62.4 pcf) or be provided with positive drainage behind the wall.

A drainage system behind the basement walls may be provided by 4-foot wide strips of Miradrain 6000 (or equivalent) placed at 8 to 10 feet on center. In our opinion, Miradrain 6000 (or equivalent), attached to the lagging and protected from the concrete placement of the walls, would provide satisfactory drainage. The Miradrain (or equivalent) strips may be placed at a depth starting at about 3 feet below the existing grade and should be connected to a perforated discharge pipe at the base of the wall. The drain pipe should consist of a minimum 4-inch-diameter perforated pipe placed with perforations down along the base of the wall.

The pipe should be sloped at least 2 inches in 100 feet and surrounded by filter gravel separated from the on-site soils by an appropriate filter fabric. The filter gravel should meet the requirements of Class 2 Permeable Material as defined in the current State of California, Department of Transportation, Standard Specifications. If Class 2 Permeable Material is not available, ¾ inch crushed rock or gravel separated from the on-site soils by an appropriate filter fabric should be used. The crushed rock or gravel should have less than 5% passing a No. 200 sieve.

The installed drainage system should be observed by personnel from our firm prior to being backfilled. Inspection of the drainage system may also be required by the reviewing governmental agencies.

Waterproofing

We recommend that all retaining walls and walls below grade be waterproofed. See Section 8.3 (Uplift and Waterproofing Considerations) for further detail.

8.5 Sulfate Attack and Corrosion Potential of Soils

Three (3) samples from the previous investigations were tested for sulfate content (results of the previous testing are included in Appendix A) and one (1) sample was tested for minimum resistivity, sulfates, chlorides, and pH during the current investigation (results of the current testing are presented in Appendix D). The corrosion tests from the current investigation were performed in accordance with the guidelines of Caltrans Test 417, 422, and 643. Based on the results of these tests, the tested soil is not considered corrosive for structures based on guidelines from the California Department of Transportation (2021).

We recommend that a corrosion consultant or project civil engineer review results of corrosion tests and provide detailed recommendations for underground metallic pipes and below-grade structures.

8.6 Excavations and Temporary Shoring

General

Earthwork operations at the site will include excavations for the subterranean parking level, removals of undocumented fill soils and rubble, excavations for foundations (potentially mats or drilled piles), and trenching for utility lines.

To provide support for the foundations, any exterior pavements, and exterior concrete walks, all existing undocumented fill soils and upper loose/soft natural soils should be excavated and replaced as engineered fill if required. Based on the understanding that the upper 15 to 20 feet of the site will be excavated for the proposed basement level, we expect that all existing fill soils will likely be removed from the site.

Temporary excavations up to a height of 4 feet can be cut vertically. Unshored excavations should not extend below a plane drawn at 1½:1 extending downward from adjacent existing footings.

Where space is available, excavations can be made with slopes of 1:1 (horizontal to vertical). Where space is unavailable, shoring is recommended for the proposed excavations adjacent to existing streets and/or buildings.

Where sloped embankments are used, the tops of the slopes should be barricaded to prevent vehicles and storage loads within 5 feet of the tops of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes; we should be advised of such heavy vehicle loadings or heavy construction equipment, stockpile material etc. so that specific setback requirements can be established. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur.

We recommend that a qualified geotechnical firm observe the excavations and shoring installation, so that necessary modifications based on variations in the soil conditions can be made. Applicable safety requirements and regulations, including OSHA regulations, should be met.

Temporary Shoring Lateral Pressures

Cantilever soldier piles or braced or tied-back shoring can be used to support the sides of the proposed excavations. For cantilever piles we recommend using the triangular lateral pressure with a maximum pressure equal to $35H$ and for the design of braced or tied-back shoring, we recommend using a trapezoidal pressure distribution with a maximum pressure equal to $24H$ (psf), where H is the retained height in feet as shown on Figure 6. All of these pressures are for level ground behind the wall (i.e. no backslope).

In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to traffic area should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the face of the shoring, the traffic surcharge may be omitted. In addition, any surcharge (live or dead load) located within a 1:1 (horizontal to vertical) plane drawn upward from the base of the shored excavation should be added to the lateral earth pressures. The details of the adjacent structures (elevation of foundation, loads, configuration, etc.) should be provided to us to estimate the pressure on the shoring walls due to surcharge, if applicable.

Design of Soldier Piles

Soldier pile shoring consists of installation of steel soldier piles in drilled holes and backfilling the holes with concrete. Lagging is placed between the soldier piles. For the design of soldier piles spaced at least 2 diameters apart on-center, the allowable lateral resistance value (passive pressure) below the bottom of the proposed excavation may be assumed to be 300 psf per foot of depth, up to a maximum of 3,000 psf for piles founded primarily in natural materials. The passive pressure includes a factor of safety of 1.5.

To develop the full lateral values, firm contact between the soldier piles and the in-situ soils must be achieved. Structural concrete should be used for that portion of the soldier pile below the bottom of the excavation. Lean-mix concrete may be used for that portion of the soldier pile below the bottom of the excavation provided that it will have sufficient strength to adequately transfer the imposed loads to the surrounding soils.

The frictional resistance between the soldier piles and the restrained earth may be taken as 500 psf. This value is based on the assumption that full bearing will be developed between the steel soldier beam and the lean-mix concrete and also between the lean-mix concrete and the retained earth.

Lagging

The soldier piles should be designed for the full anticipated lateral pressure; however, the pressure on the lagging will be less due to arching effects in the soils. Therefore, the lagging can be designed for the recommended earth pressure but limited to a maximum value of 400 psf. Careful installation of the lagging will be necessary to achieve bearing against the retained earth.

Tie-Back Anchor Design

Tieback friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees from the vertical through the bottom of the excavation. These anchors should extend to a minimum of 15 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

For preliminary design purposes, it may be estimated that drilled and grouted friction anchors would develop a soil friction of 750 psf along the anchors in the bonded zone. This value is provided for gravity grouted anchors. For pressure grouted anchors, a soil friction of 2,500 psf may be used along the anchors in the bonded zone. The capacities of the anchors should be determined by testing of the initial anchors as outlined below under the Tie-back Anchor Testing section.

Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6-feet on center, then no reduction in capacity is necessary. Closer spacing would require evaluation of an appropriate reduction factor.

Tie-Back Anchor Installation

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. The anchors should be filled with concrete, placed by pumping from the tip out. The concrete should extend from the tip of the anchor to the active wedge. To minimize caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand. A small amount of cement may be used to allow the sand to be placed by pumping. The sand-cement mixture should fill the portion of the tieback anchor tightly and should be flush with the face of the shoring when finished.

Tie-Back Anchor Testing

The installation of the anchors and the testing of the completed anchors should be observed by a representative of a qualified geotechnical firm. The geotechnical engineer or his representative should select at least four of the initial anchors for 24-hour 200% tests and 10% of the total number of anchors for "quick" 200% tests to verify in the field the friction value assumed in this report. Also,

we recommend that the 200% tests be performed at representative locations around the site and not concentrated in a single area.

The total deflection during the 24-hour 200% tests should not exceed 12 inches during loading; the anchor deflection should not exceed $\frac{3}{4}$ inch during the 24-hour period, measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than $\frac{1}{2}$ inch, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

For the quick 200% tests, the test load should be maintained for 30 minutes. The total deflection of the anchor during the 200% quick test should not exceed 12 inches; the deflection after the 200% test load has been applied should not exceed $\frac{1}{4}$ inch during the 30-minute period.

All of the production anchors should be pre-tested to at least 150% of the design load; the total deflection during the test should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15-minute period in order for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked off at the design load. The locked-off load should be verified by rechecking the load on the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by a qualified geotechnical firm.

Raker Bracing

Raker bracing, if used, should be supported by temporary concrete footings (deadmen). For design of such temporary footings, poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 1,500 pounds per square foot may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade and is founded in the native alluvium. To reduce the deflection of the shoring, the rakers should be preloaded to the design load.

Deflection

Predicting actual deflections of a shored embankment is difficult. It should, however, be realized that some deflection would occur. We estimate that deflections could be about 1 inch at the top of the

shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to prevent settlement and loss of support from beneath and adjacent to the shored excavation.

Monitoring

Monitoring of the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles. Initial survey should be taken prior to the first level of excavation so that an accurate baseline may be established.

We recommend that the initial survey and monitoring program also include any adjacent existing structures. Photographs and videos of the existing structures are recommended as part of the documentation process.

Monitoring considerations should be discussed further with the design consultants and the contractor when the design of the shoring system has been finalized.

8.7 Earthwork

General

Earthwork should be performed in accordance with the applicable sections of the grading code for the City of Los Angeles and the State of California, as well as the recommendations in this report.

Subgrade Preparation and Moisture Conditioning

Areas excavated to receive fill should be cleared and stripped of all debris, deleterious matter, organics and vegetation, and remnants resulting from demolition of existing foundations. Cleared and grubbed material should be disposed of offsite.

After clearing the site of existing debris, the exposed subgrade should be observed for debris, organic material, or other undesirable materials. The exposed subgrade should then be proof-rolled so as to allow placement of any required fill. Compacted fill should be placed immediately upon approval of the prepared subgrade by the geotechnical engineer of record.

Mat/Foundation Excavations

The exposed excavated surface should be observed by the geotechnical engineer to confirm that satisfactory subgrade soils have been encountered. If loose, soft or clayey native soils, or undocumented fill soils are encountered at the bottom of excavation, additional removals may be required. The bottom of the excavations should be proof-rolled so as to allow placement of any required fill at 95% relative compaction in accordance with ASTM D1557, or the placement of concrete

or concrete slurry mix as backfill. Compacted fill should be placed immediately upon approval of the prepared subgrade by the geotechnical engineer of record.

Where foundation excavations are deeper than about 4 feet, the sides of the excavations should be sloped back at $\frac{3}{4}$:1 (horizontal to vertical) or shored for safety. Unshored excavations should not extend below a plane drawn at $1\frac{1}{2}$:1 (horizontal to vertical) extending downward from adjacent existing foundations.

Fill Materials and Placement of Fill

The on-site excavated granular materials such as sands and silty sands can be used as engineered fill. However, the on-site clayey soils are anticipated to be moderately expansive and should not be used within 3 feet of the lightly-loaded foundation, slabs or pavements. The existing fill materials, once debris and vegetation is removed, may be re-used as compacted fill. Oversized material (greater than 6 inches in longest dimension) should be removed from excavated material prior to reuse as engineered fill.

Imported fill material should be granular, non-corrosive, free of organic matter or other deleterious material. The Expansion Index of the fill material should be less than 35 and fill material should have a fines content (passing #200 sieve) less than 40 percent. Oversize material (larger than 6 inches in diameter) should not be used in the fill. All imported fill material should be approved by the geotechnical engineer prior to placement. A sample of proposed fill material(s) should be submitted to the geotechnical engineer for testing at least three business days prior to use at the site.

Fill material should be compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction.

Backfill

All required backfill should be mechanically compacted in layers; flooding should not be permitted. Proper compaction of backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying elements such as slabs and paving. Backfill should be compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction. The on-site soils excluding clayey soils may be used in the compacted backfill.

Some settlement of the backfill should be expected, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the building. Also, provisions should be made for some settlement of concrete walks supported on backfill.

The exterior grades should be sloped to drain away from the foundation to prevent ponding of water.

Compaction

The preparation of the subgrade, excavations for the mat foundation and reworking of on-site soils and compaction of any required fills or backfill should be observed and tested by a representative of a qualified geotechnical firm.

The bottom of the excavations should be proof-rolled so as to allow placement of any required fill at 95% relative compaction in accordance with ASTM D1557. Compacted fill should be placed immediately upon approval of the prepared subgrade by the geotechnical engineer of record.

Any required fill below the foundations should be compacted to a minimum of 95 percent maximum dry density as determined in accordance with ASTM D 1557. The field density of fill should be determined in accordance with the Sand Cone Method (ASTM D1556) or the Nuclear Method (ASTM D2922 and D3017).

Fill material should be placed in loose lifts generally no greater than 8 inches thick. The moisture content of the on-site sandy soils at the time of compaction should vary no more than 2% below or above optimum moisture content. The moisture content of the on-site clayey soils at the time of compaction should be between 2% and 4% above optimum moisture content.

8.8 Geotechnical Observation

We recommend that a qualified geotechnical engineer or his representative observe the condition of the final subgrade soils immediately prior to foundation construction, and if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade. This representative should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the installation of shoring and tie-backs and testing of tie-back anchors.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The representative should also observe proof-rolling and delineation of areas requiring over-excavation.
- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.



- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

9.0 GENERAL CONDITIONS

In view of the general geology of the project area, the possibility of different subsurface conditions cannot be discounted. Conclusions and recommendations presented in this report are based upon GeoPentech's understanding of the project and the assumption that the subsurface conditions do not deviate appreciably from those disclosed by the field explorations performed. In the event that the locations, configurations, layout, or features of the proposed tower and associated podium are changed, the recommendations presented in this report may not be applicable. It is the responsibility of the Owner to bring any such changes of the proposed structures and any deviations of the subsurface conditions to the attention of GeoPentech. In this way, supplemental recommendations, if required, can be made without delay to the project.

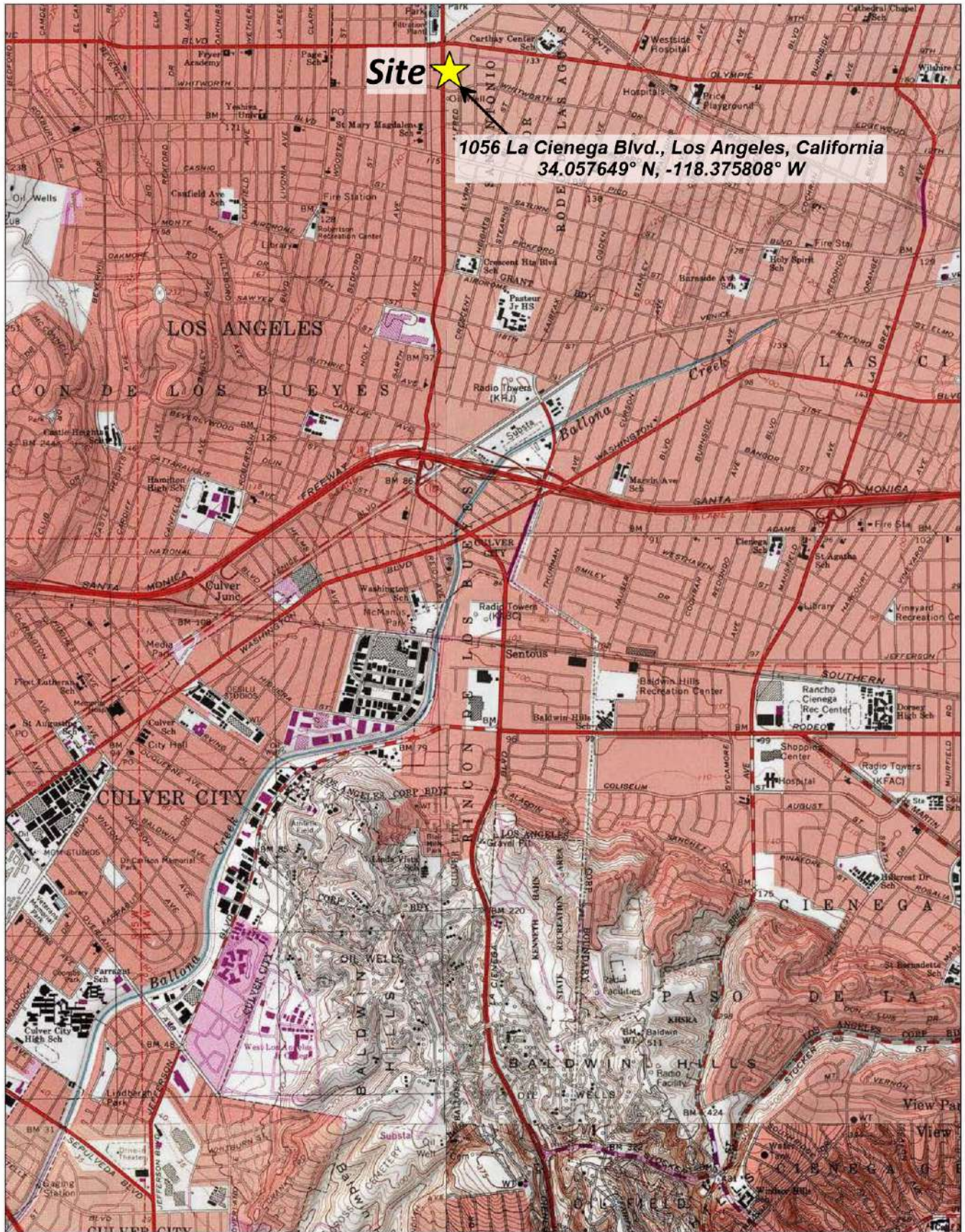
Professional judgments presented in this report are based on an evaluation of the technical information gathered and GeoPentech's general experience in the field of geotechnical engineering. GeoPentech does not guarantee the performance of the project in any respect, only that the engineering work and judgment rendered meet the standard of care of the geotechnical profession at this time.

10.0 REFERENCES

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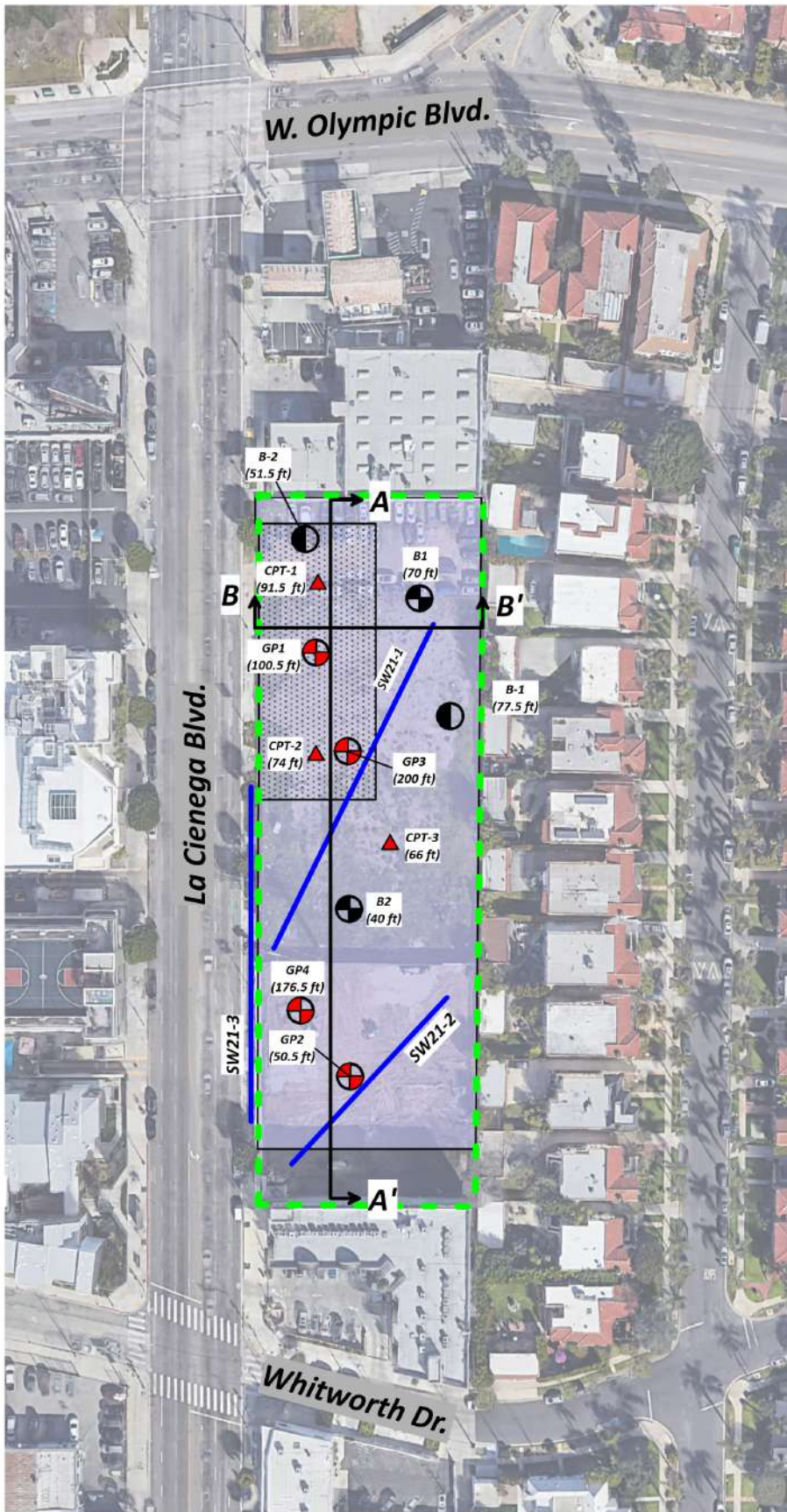
SITE LOCATION MAP

Project No.: 21086A










Project: 1056 LA CIENEGA BLVD.

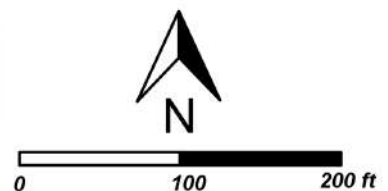
Date: SEP 2021

Figure 1

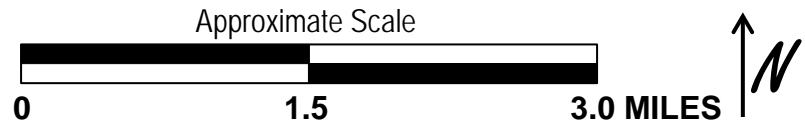


Explanations

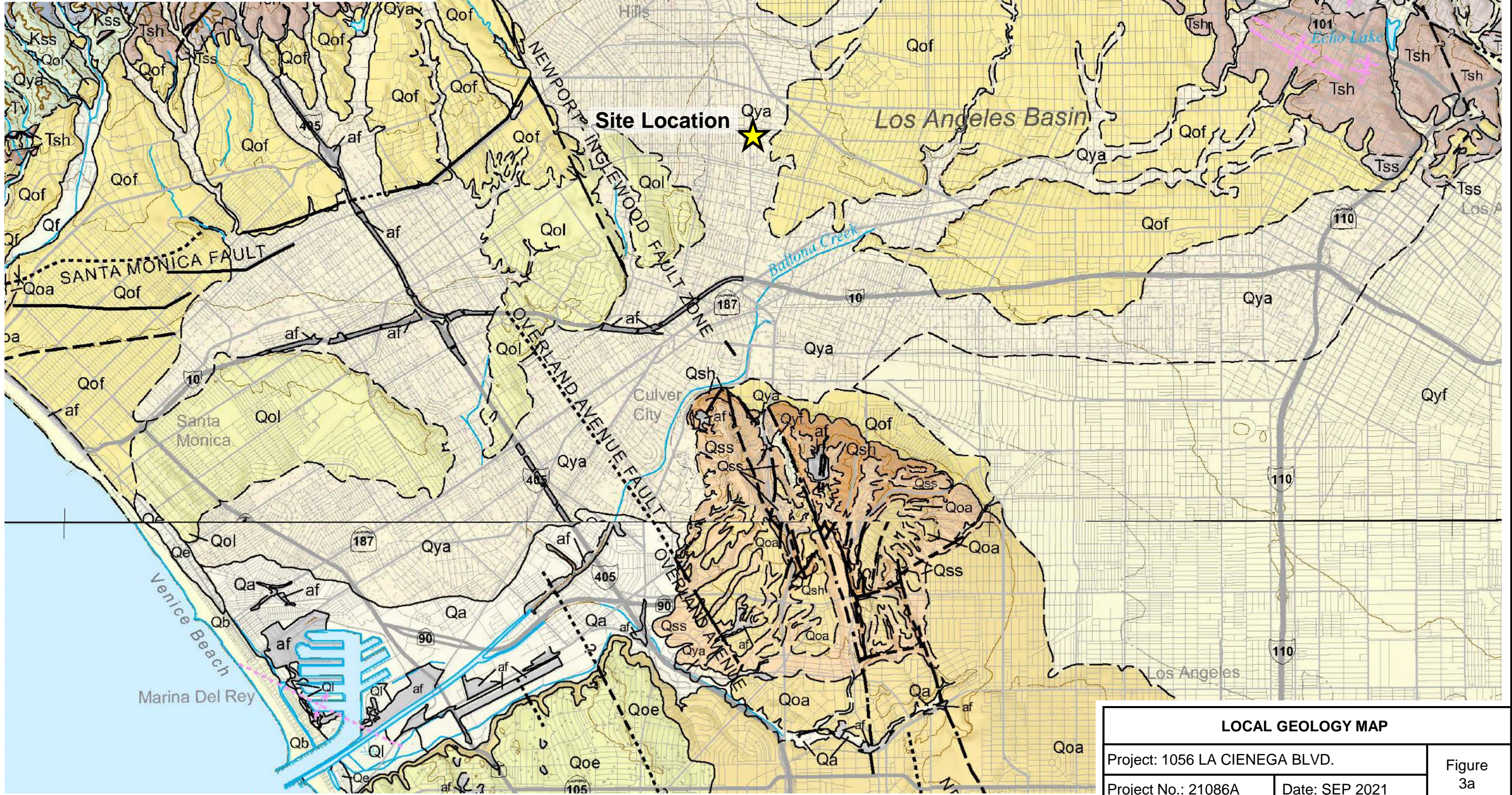
-  Approximate Extent of Project Site
-  **B1** (50 ft) Approximate Location of Previous Borings by Geotechnologies (2012)
-  **B-1** (50 ft) Approximate Location of Previous Borings by AGI Geotechnical (2009)
-  **GP1** (50 ft) Approximate Location of Borings of Current Investigation
-  Approximate Location of Cone Penetration Tests
-  **SW21-1** MASW Lines
-  **A A'** Cross Sections
-  Approximate Extent of Tower Structure
-  Approximate Extent of Podium Structure



SITE PLAN AND BORING LOCATIONS



Source: CGS (2012), compiled by Bedrossian, T.L., and Roffers, P.D., Geologic Compilations of Quaternary Surficial Deposits in Southern California, Los Angeles 30' x 60' Quadrangle (Revised):CGS Special Report 217, Plate 9, scale 1:100,000.













LOCAL GEOLOGY MAP		
Project: 1056 LA CIENEGA BLVD.		Figure 3a
Project No.: 21086A	Date: SEP 2021	



MAP UNITS

Source: CGS (2012), compiled by Bedrossian, T.L., and Roffers, P.D., Geologic Compilations of Quaternary Surficial Deposits in Southern California, Los Angeles 30' x 60' Quadrangle (Revised):CGS Special Report 217, Plate 9, scale 1:100,000.





Late Holocene (Surficial Deposits)

-  **Artificial Fill** - deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills
-  **Undifferentiated Surficial Deposits** - includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers
-  **Landslide Deposits** - may include debris flows and older landslides of various earth material and movement types; unconsolidated to moderately well-consolidated
-  **Beach Deposits** - unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand
-  **Alluvial Wash Deposits** - unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand
-  **Alluvial Fan Deposits** - unconsolidated boulders, cobbles, gravel, sand, and silt recently deposited where a river or stream issues from a confined valley or canyon; sediment typically deposited in a fan-shaped cone; gravelly sediment generally more dominant than sandy sediment
-  **Alluvial Valley Deposits** - unconsolidated clay, silt, sand, and gravel recently deposited parallel to localized stream valleys and/or spread more regionally onto alluvial flats of larger river valleys; sandy sediment generally more dominant than gravelly sediment
-  **Terrace Deposits** - includes marine and stream terrace deposits; marine deposits include slightly to moderately consolidated and bedded gravel and conglomerate, sand and sandstone, and silt and siltstone; river terrace deposits consist of unconsolidated thin- to thick-bedded gravel
-  **Lacustrine, Playa, and Estuarine (Paralic) Deposits** - mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites
-  **Eolian and Dune Deposits** - unconsolidated, generally well-sorted wind-blown sand; may occur as dune forms or sheet sand



Holocene to Late Pleistocene (Surficial Deposits)

-  **Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
-  **Young Alluvial Valley Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers



Late to Middle Pleistocene (Surficial Deposits)

-  **Old Alluvial Fan Deposits** - slightly to moderately consolidated, moderately dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
-  **Old Alluvial Valley Deposits** - slightly to moderately consolidated, moderately dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers
-  **Old Terrace Deposits** - slightly to moderately consolidated, moderately dissected marine and stream terrace deposits
-  **Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types

Middle to Early Pleistocene (Surficial Deposits)

-  **Very Old Alluvial Fan Deposits** - moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
-  **Very Old Alluvial Valley Deposits** - moderately to well-consolidated, highly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers; generally uplifted and deformed






Quaternary (Bedrock)

-  **Coarse-grained formations of Pleistocene age and younger** - primarily sandstone and conglomerate
-  **Fine-grained formations of Pleistocene age and younger** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments

Tertiary (Bedrock)














-  **Coarse-grained Tertiary age formations** - primarily sandstone and conglomerate
-  **Fine-grained Tertiary age formations** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments
-  **Tertiary age formations of volcanic origin**

Mesozoic and Older (Bedrock)

-  **Coarse-grained Cretaceous age formations of sedimentary origin**
-  **Fine-grained Cretaceous age formations of sedimentary origin**
-  **Cretaceous and pre-Cretaceous metamorphic formations of sedimentary and volcanic origin**
-  **Serpentinite of all ages**
-  **Granitic and other intrusive crystalline rocks of all ages**

SYMBOL EXPLANATION

[For geologic line symbols: lines are solid where location is accurate, long-dashed where location is approximate, short-dashed where location is inferred, dotted where location is concealed. Queries added where identity or existence may be questionable.]

- Contacts**
-  Contact
 -  Gradational contact
 -  Reference contact -- Used to delineate geologic units that were mapped as separate units on the original source map, but are consolidated on this map.
 -  Fault -- Includes strike-slip, normal, reverse, oblique, and unspecified slip
 -  Lineament
- Folds -- Showing direction of plunge where appropriate**
-  Anticline
 -  Overturned anticline
 -  Syncline
-  Dike
 -  Stream
 -  Spring
 -  Road
 -  County boundary

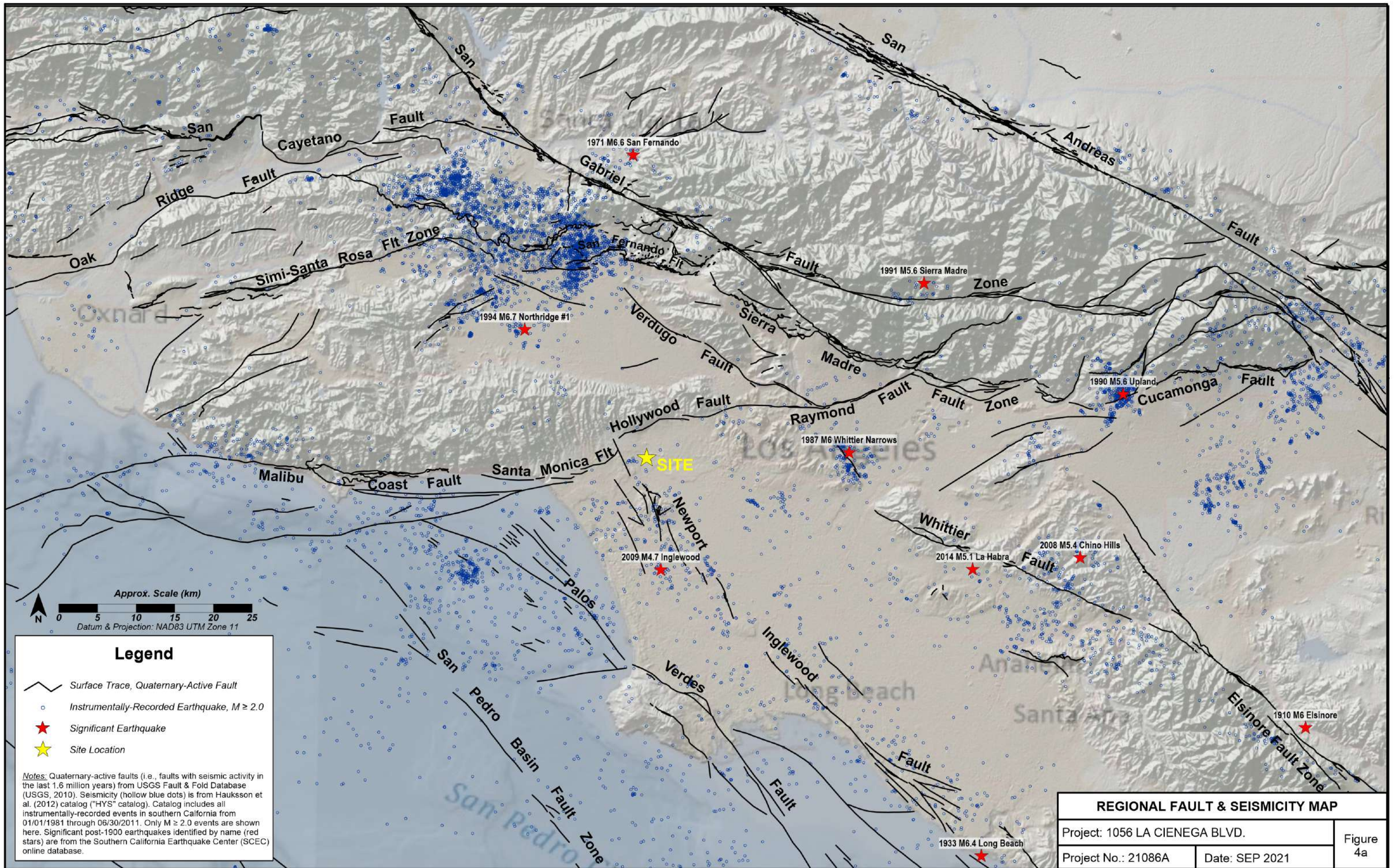
LOCAL GEOLOGY MAP LEGEND

Project: 1056 LA CIENEGA BLVD.

Figure
3b

Project No.: 21086A

Date: SEP 2021

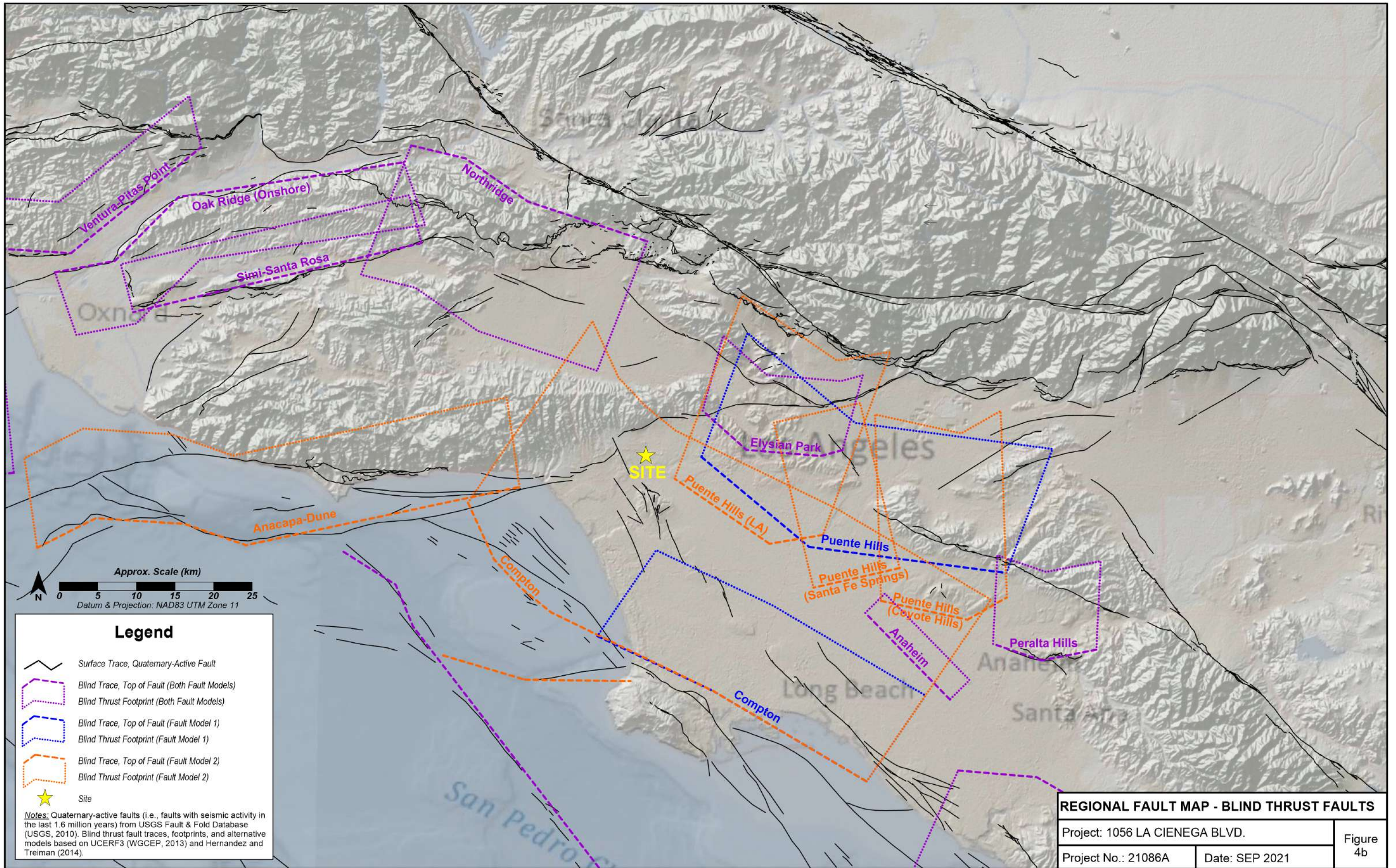


Legend

- Surface Trace, Quaternary-Active Fault
- Instrumentally-Recorded Earthquake, $M \geq 2.0$
- Significant Earthquake
- Site Location

Notes: Quaternary-active faults (i.e., faults with seismic activity in the last 1.6 million years) from USGS Fault & Fold Database (USGS, 2010). Seismicity (hollow blue dots) is from Hauksson et al. (2012) catalog ("HYS" catalog). Catalog includes all instrumentally-recorded events in southern California from 01/01/1981 through 06/30/2011. Only $M \geq 2.0$ events are shown here. Significant post-1900 earthquakes identified by name (red stars) are from the Southern California Earthquake Center (SCEC) online database.

REGIONAL FAULT & SEISMICITY MAP		
Project: 1056 LA CIENEGA BLVD.		Figure 4a
Project No.: 21086A	Date: SEP 2021	



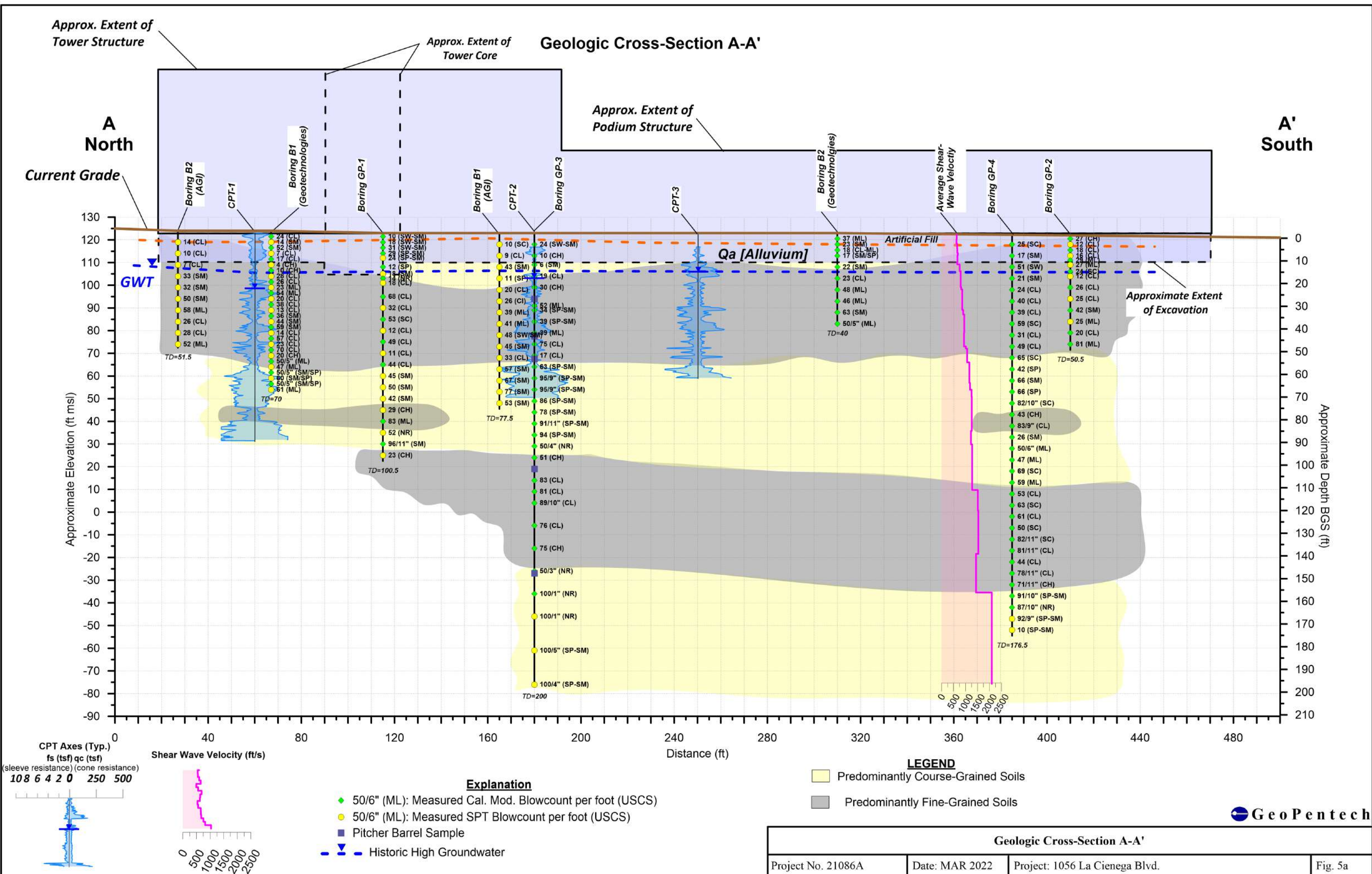
REGIONAL FAULT MAP - BLIND THRUST FAULTS

Project: 1056 LA CIENEGA BLVD.		Figure 4b
Project No.: 21086A	Date: SEP 2021	

Legend

- Surface Trace, Quaternary-Active Fault
- Blind Trace, Top of Fault (Both Fault Models)
- Blind Thrust Footprint (Both Fault Models)
- Blind Trace, Top of Fault (Fault Model 1)
- Blind Thrust Footprint (Fault Model 1)
- Blind Trace, Top of Fault (Fault Model 2)
- Blind Thrust Footprint (Fault Model 2)
- Site

Notes: Quaternary-active faults (i.e., faults with seismic activity in the last 1.6 million years) from USGS Fault & Fold Database (USGS, 2010). Blind thrust fault traces, footprints, and alternative models based on UCERF3 (WGCEP, 2013) and Hernandez and Treiman (2014).



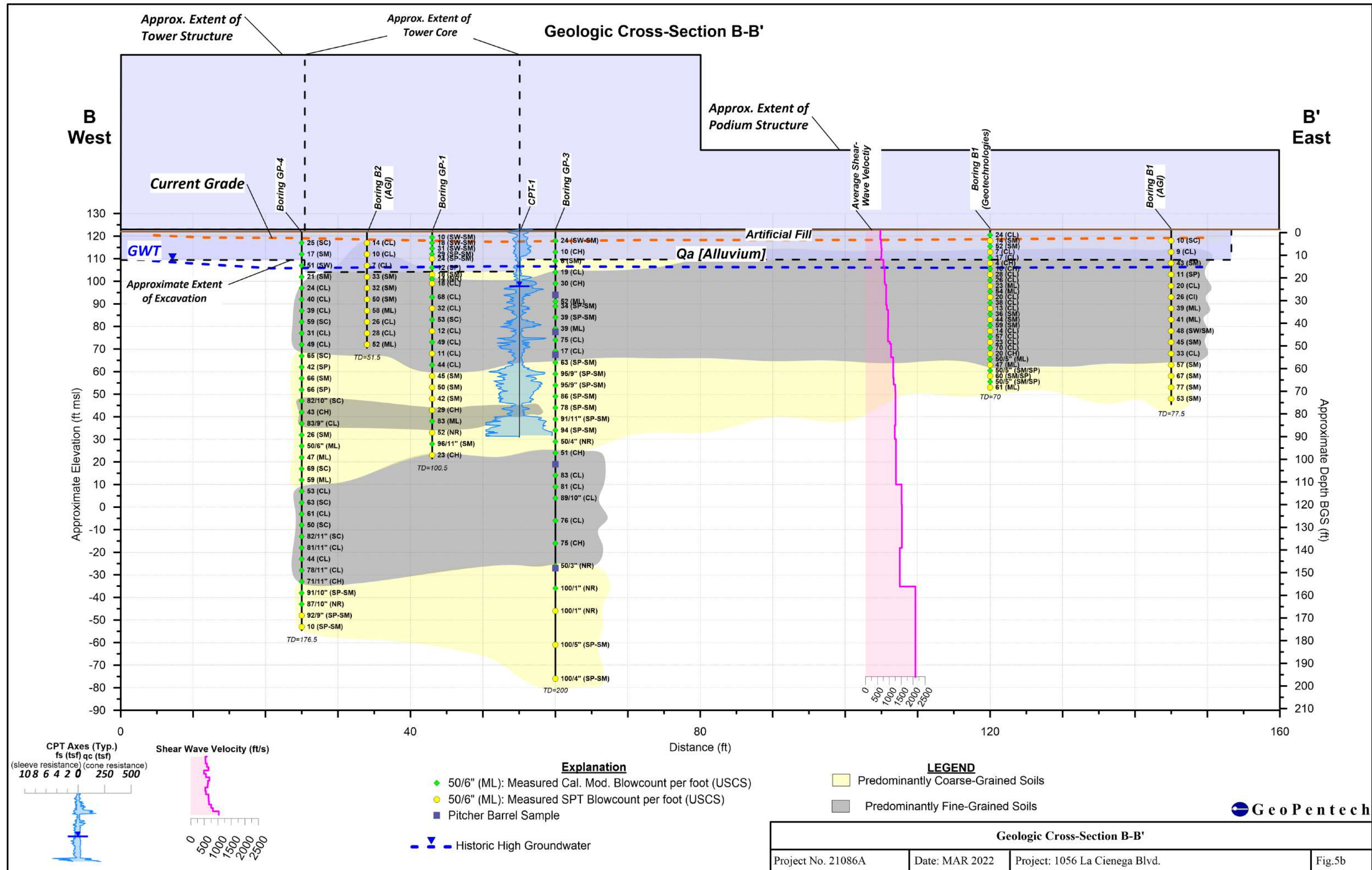
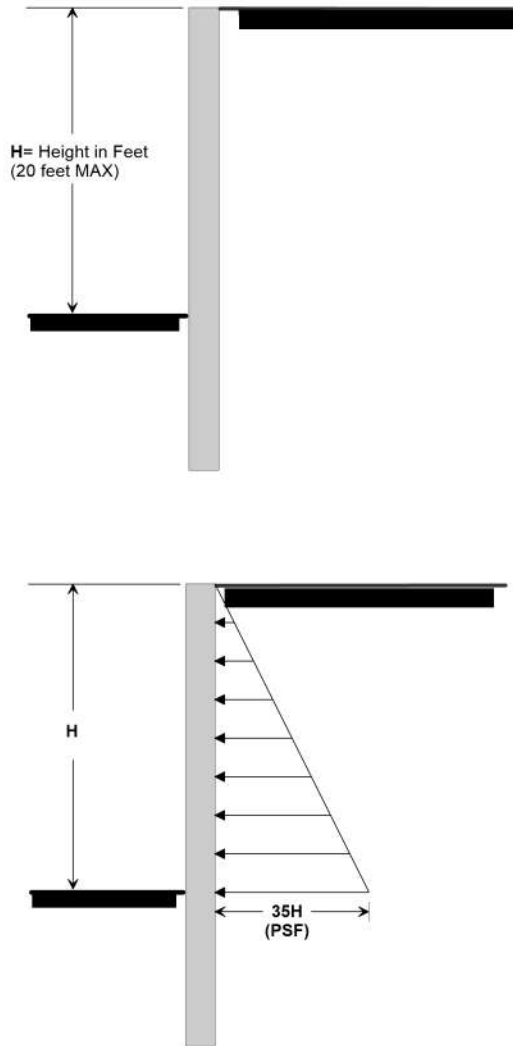
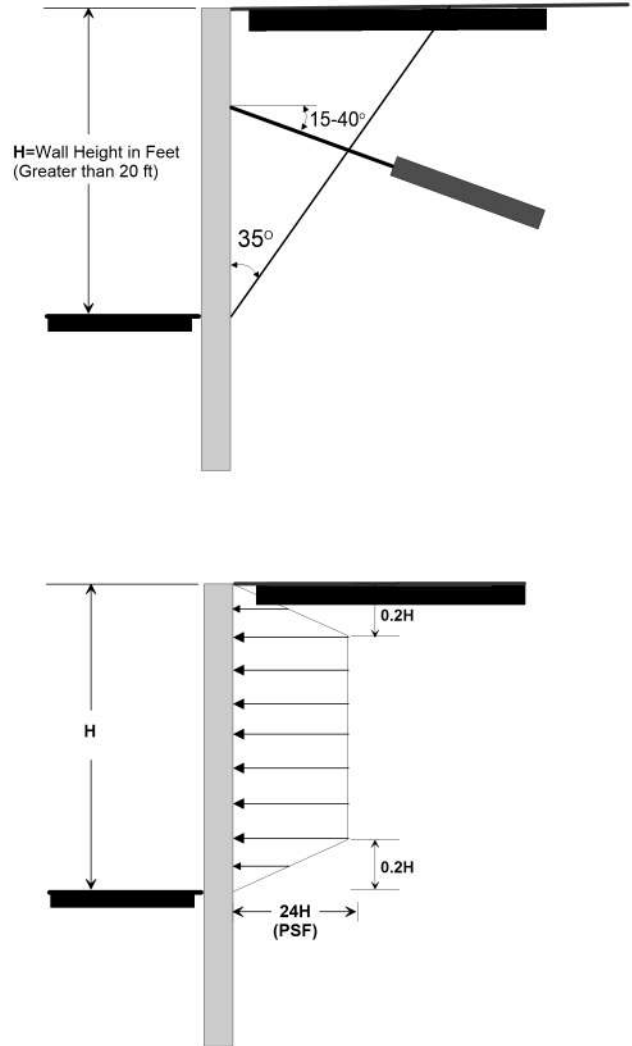


Figure 6a
Cantilever Shoring



1. Lateral Earth Pressure is $35H$ with a triangular distribution.
2. The lateral pressure above assumes no build up of hydrostatic pressure behind the shoring.
3. **Soldier Pile Design** - Spaced at 2 diameters on center. Maximum tributary area of 3 pile diameters. Passive pressure below bottom of excavation is 300 psf per foot of depth up to a maximum value of 3,000 psf. Frictional resistance between soldier pile and soil below excavation is 500 psf.
4. **Lagging** - Design for full anticipated lateral pressure indicated above, but limit to a maximum value of 400 psf.
5. The shoring should be designed to resist applicable surcharge loads from adjacent structures, stockpiled material, or traffic loads.

Figure 6b
Tiedback Shoring



1. Maximum Lateral Earth Pressure is $24H$ with a trapezoidal distribution.
2. **Soldier Pile Design** - Spaced at 2 diameters on center. Maximum tributary area of 3 pile diameters. Passive pressure below bottom of excavation is 300 psf per foot of depth up to a maximum value of 3,000 psf. Frictional resistance between soldier pile and soil below excavation is 500 psf.
4. **Lagging** - Design for full anticipated lateral pressure indicated above, but limit to a maximum value of 400 psf.
5. **Tie-Back Anchor** - Friction of 750 psf (for gravity grouted anchors) or 2,500 psf (for pressure grouted anchors) along the anchors in bonded zone.
6. The shoring should be designed to resist applicable surcharge loads from adjacent structures, stockpiled material, or traffic loads.

TEMPORARY SHORING DETAILS

APPENDIX A

PREVIOUS BORING LOGS

Borings Performed By:
CalWest Geotechnical
(2008)

EXCAVATION DATA

PROJECT SRC/SCH LLC

JOB NO: G 4869

EXCAVATION METHOD Screw Auger

DATE: Dec., 06

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	BLOW COUNTS PER 6"	DEPTH IN FEET	DESCRIPTION: BB-1
2.5'	R	CL	17.4	102.3		1	0-5' SOIL: Silty Clay with gravel, grayish brown, moist, medium stiff to stiff.
						2	
						3	
						4	
5'	R	CL	19.2	100.1		5	5' - 10' SOIL: Silty Clay, grayish brown to dark brown, moist, stiff.
						6	
						7	
						8	
10'	R	CL	21.8	97.2		9	10' - 15' SOIL: Silty Clayey and Sandy Clay, grayish orange brown, moist, stiff.
						10	
						11	
						12	
15'	R	CL/CH	22.2	99.8		13	15' - 20' SOIL: Silty Clay, medium gray to dark gray, moist, stiff. Groundwater at 17 feet.
						14	
						15	
						16	
20'	R	CL/CH	23.8	100.7		17	20' - 25' SOIL: Silty Clay, medium gray, moist, stiff.
						18	
						19	
						20	
						21	END at 25' Groundwater at 17' No Caving
						22	
						23	
						24	
						25	
						26	
						27	
						28	
						29	
						30	

EXCAVATION DATA

PROJECT SRC/SCH LLC

JOB NO: G 4869

EXCAVATION METHOD Screw Auger

DATE: Dec., 06

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	BLOW COUNTS PER 6"	DEPTH IN FEET	DESCRIPTION: BB-2
2.5'	R	CL	19.3	105.6		1	0-5' SOIL: Silty Clay with gravel, dark brown, moist, medium stiff to stiff.
						2	
						3	
						4	
5'	R	CL	17.7	101.3		5	5' - 10' SOIL: Sandy Clay, reddish brown to dark brown, moist, stiff.
						6	
						7	
						8	
10'	R	CL	21.6	96.8		9	10' - 15' SOIL: Silty Clayey and Sandy Clay, grayish brown, moist, stiff.
						10	
						11	
						12	
15'	R	CL/CH	21.8	101.2		13	15' - 20' SOIL: Silty Clay, bluish gray to dark gray, moist, stiff. Groundwater at 17.5 feet.
						14	
						15	
						16	
20'	R	CL/CH	22.8	102		17	20' - 25' SOIL: Sandy Clay, bluish gray, moist, stiff.
						18	
						19	
						20	
						21	
						22	
						23	
						24	
						25	
						26	
						27	END at 25'
						28	Groundwater at 17.5'
						29	No Caving
						30	

EXCAVATION DATA

PROJECT FREIM

JOB NO: G 4088

EXCAVATION METHOD 6" Dia. HALLOW STEM AUGER

DATE: APRIL, 04

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	BLOW COUNTS PER 6"	DEPTH IN FEET	DESCRIPTION: B-1
2.5'	R	CL	18.5	104.4		1	0 - 2.5' 3" Asphalt over: SOIL: Silty Sandy Clay, dark grey, moist, stiff.
						2	
						3	
						4	
5'					8,11,13	5	2.5' - 5' SOIL: Sandy Clay, medium brown, moist, stiff.
						6	
						7	
7.5'	R	CL	19.4	98.6		8	5' - 10' SOIL: Silty Clay with gravel, greyish brown, moist, stiff. A thin lense of Clayey Sand with Gravel at seven feet.
						9	
						10	
10'					7,9,12	11	
						12	
12.5'	R	CL/CH	26.2	96.2		13	10' - 15' SOIL: Silty Clay, greyish brown, moist, stiff.
						14	
						15	
15'					8,10,19	16	
						17	
17.5'	R	CL/SC	23.1	101.7		18	15' - 20' SOIL: Silty Clay with a thin layer of Clayey Sand with gravel at 17feet, greyish brown, moist, dense to stiff. Groundwater at 17.5 feet.
						19	
						20	
20'					7,9,12	21	
						22	
22.5'	R	CL	19.5	106.7		23	20' - 25' SOIL: Silty Clay, greyish brown, moist, stiff.
						24	
						25	
25'					15,19,21	26	
						27	
27.5'	R	CL	17.3	107.8		28	25' - 30' SOIL: Fine Silty Clay, greyish brown, moist, stiff.
						29	
30'					15,23,20	30	

REFERENCE: C:\OWINFILE\LAB\LOG.XLS

EXCAVATION DATA

PROJECT FREIM

JOB NO: G4088

EXCAVATION METHOD 6" dia. HALLOW STEM AUGER

DATE: APRIL, 04

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	BLOW COUNTS PER 6"	DEPTH IN FEET	DESCRIPTION:
32.5'	R	CL	21.2	104.6		31	30' - 35' SOIL: Silty Clay, dark grey, fine grained, moist, stiff.
						32	
35'					9,14,22	33	35' - 40' SOIL: Silty Clay, dark grey, fine grained, moist, stiff.
						34	
40'					11,16,18	35	40' - 45' SOIL: Sandy Silt, bluish grey, fine grained, moist, stiff.
						36	
45'					12,18,24	37	45' - 50' SOIL: Silty Clay, dark grey, fine grained, moist, stiff.
						38	
50'					10,16,27	39	
						40	
						41	
						42	
						43	
						44	
						45	
						46	
						47	
						48	
						49	
						50	

END @ 50'
GROUNDWATER @ 17.5'
NO CAVING

REFERENCE: C:\OWINFILE\LAB\LOG.XLS

EXCAVATION DATA

PROJECT FREIM

JOB NO: G4088

EXCAVATION METHOD BACKFILL 11a. HALLOW STEM AU

DATE: APRIL, 04

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	RELATIVE DENSITY (%)	DEPTH IN FEET	DESCRIPTION:
0'	R	CL	18.1	105.3		1	0- 5' 3" Asphalt over: SOIL: Silty Sandy Clay, greyish brown, moist, stiff.
						2	
						3	
						4	
5'	R	CL	21.6	100.2		5	5' - 10' SOIL: Silty Clay, greyish brown, moist, stiff.
						6	
						7	
10'	R	CL/CH	26.5	96.4		10	10' - 15' SOIL: Silty Sandy Clay, greyish brown, fine grained, stiff.
						11	
						12	
15'	R	CL/SC	15.2	107.8		15	15' - 20' SOIL: Silty Clay with a thin layer of Clayey Sand with gravel at 17 feet, greyish brown to yellowish brown, moist, dense, to stiff. Groundwater at 17.5 feet.
						16	
						17	
						18	
20'	R	CL	22	103.1		20	<p>END @ 20'</p> <p>GROUNDWATER @ 17.5'</p> <p>NO CAVING</p>
						21	
						22	
						23	
						24	
						25	
						26	
						27	
						28	
						29	
						30	

REFERENCE: C:\0TEMPLT\SOILS\LAB\LOG.XLS

EXCAVATION DATA

PROJECT DIDM Development Corp.

JOB NO: G 4158

EXCAVATION METHOD 6" Dia. HALLOW STEM AUGER

DATE: November, 04

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	BLOW COUNTS PER 6"	DEPTH IN FEET	DESCRIPTION:
2.5'	R	CL	19.2	103.4		1	0 - 5' 3" Concrete cover; SOIL: Silty Clay with gravel, dark brown to greyish brown, moist, stiff.
						2	
						3	
						4	
5'					6,8,12	5	5' - 10' SOIL: Silty Clay with gravel, greyish brown, moist, stiff.
						6	
7.5'	R	CL	19.4	99.8		7	
						8	
10'					7,16,24	9	10' - 15' SOIL: Silty Clay, greyish brown, moist, stiff.
						10	
12.5'	R	CL/CH	21.3	97.8		11	
						12	
15'					5,9,19	13	15' - 20' SOIL: Sandy Clayey Silt, medium brown, fine grained, moist, stiff. Groundwater at 17 feet.
						14	
17.5'	R	CL	18.5	103.2		15	
						16	
20'					10,11,16	17	20' - 25' SOIL: Silty Clay, greyish brown, moist, stiff.
						18	
22.5'	R	CL	20.1	102.6		19	
						20	
25'					6,11,14	21	25' - 30' SOIL: Fine Silty Clay, greyish brown, moist, stiff.
						22	
27.5'	R	CL	18.5	105.8		23	
						24	
30'					16,17,16	25	
						26	
						27	
						28	
						29	
						30	

REFERENCE: C:\COWIN\FILE\LAB\LOG.XLS

EXCAVATION DATA

PROJECT DIDM Development Corp.

JOB NO: G4158

EXCAVATION METHOD 6" dia. HALLOW STEM AUGER

DATE: November, 04

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	BLOW COUNTS PER 6"	DEPTH IN FEET	DESCRIPTION:
32.5'	R	CL	20.6	102.7		31 32 33	30' - 35' SOIL: Silty Sandy Clay, dark grey, fine grained, moist, stiff.
35'					14, 20, 30	34 35 36	
						37 38	
40'					14, 22, 30	39 40 41	35' - 40' SOIL: Silty Clay, dark grey, fine grained, moist, stiff.
42.5'	R	CL	18.8	104.6		42	40' - 45' SOIL: Sandy Silty Clay, bluish grey, fine grained, moist, stiff.
45'					18, 18, 27	43 44 45	
						46 47	
50'					12, 23, 28	48 49 50	45' - 50' SOIL: Silty Clay, dark grey, fine grained, moist, stiff.

END @ 50'
GROUNDWATER @ 17.0'
NO CAVING

REFERENCE: C:\OWINFILE\LAB\LOG.XLS

Borings Performed By:

AGI Geotechnical

(2009)



A.G.I. GEOTECHNICAL, INC.

AGI Geotechnical, Inc. 16555 Sherman Way, Suite A Van Nuys, California 91406 Telephone: (818) 785-6244 Fax: (818) 785-8251

CLIENT DIDM Development Corp. PROJECT NAME Five Story Apartment over Two Level Parking
 PROJECT NUMBER 19-3342-00 PROJECT LOCATION 1022-1054 S. La Cienega Blvd.
 DATE STARTED 7/17/2009 COMPLETED 7/17/2009 GROUND ELEVATION _____ TEST PIT SIZE _____
 EXCAVATION CONTRACTOR Chioce Drilling GROUND WATER LEVELS: _____ POINT COORDINATES: _____
 EXCAVATION METHOD 8" Hollow stem Auger AT TIME OF EXCAVATION 15' NORTH(Y) _____
 LOGGED BY W.A.F. CHECKED BY J.A.V. AT END OF EXCAVATION 17' SOUTH(X) _____
 NOTES _____ AFTER EXCAVATION 17' ELEV(Z) _____

DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOW COUNT	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	ATTERBERG LIMITS			MATERIAL DESCRIPTION	<200	D ₅₀	USCS GROUP SYMBOL
						LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX				
5.0			17/20	11.3	99.9				Dark Brown Sandy CLAY (Moist, Firm)	60	.02	CL
7.5			4/5/5 (10)			28	19	9	Brown Clayey SAND with Scattered Gravel (Moist, Medium Dense)	40	.17	SC
10.0			7/8	11.1	107.8				Brown Sandy CLAY (Very Moist, Stiff)	61	.02	CL
12.5			3/4/5 (9)									
15.0			8/19	18.9	108.0				Brown Silty SAND with Gravel (Moist, Dense)	13	1.05	SM
17.5			19/20 /23 (43)									
20.0			7/8	13.0	114.9				Gray Brown Coarse SAND and Gravel (Moist, Dense)	26	.28	SP
22.5			3/5/6 (11)			37	26	11	Brown Silty CLAY (Very Moist, Stiff)	96	.01	CL
25.0			6/14	29.1	96.5				Brown Sandy CLAY (Wet, Very Stiff)	98	.02	CL
27.5			6/9/1 (20)									
30.0			15/25	22.1	108.0				Gray Brown Sandy CLAY (Very Moist, Hard)	74	.03	CL
32.5			8/11 /15 (28)			40	25	15				
35.0			20/35	24.8	99.5				Gray Sandy SILT (Very Moist, Dense)	58	.07	ML
37.5			10/16 /23 (39)									
40.0			20/30	18.6	110.9				Gray Sandy SILT - Interbedded Gray Fine to Coarse SAND (Wet, Dense)			ML
42.5			8/19 /22 (41)						Gray Well Graded SAND with SILT (Wet, Dense)	12	.07	SW/SM



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AGI Geotechnical, Inc. 16555 Sherman Way, Suite A Van Nuys, California 91408 Telephone: (818) 785-6244 Fax: (818) 785-8261

CLIENT <u>DIDM Development Corp.</u>	PROJECT NAME <u>Proposed Residence</u>
PROJECT NUMBER <u>19-3342-00</u>	PROJECT LOCATION <u>Parcel 1, Jim Bridger Road</u>
DATE STARTED <u>7/17/2009</u> COMPLETED <u>7/17/2009</u>	GROUND ELEVATION _____ TEST PIT SIZE _____
EXCAVATION CONTRACTOR <u>Choice Drilling</u>	GROUND WATER LEVELS: _____ POINT COORDINATES: _____
EXCAVATION METHOD <u>8" Hollow stem Auger</u>	AT TIME OF EXCAVATION <u>15'</u> NORTH/(Y) _____
LOGGED BY <u>W.A.F.</u> CHECKED BY <u>J.A.V.</u>	AT END OF EXCAVATION <u>17'</u> SOUTH/(X) _____
NOTES _____	AFTER EXCAVATION <u>17'</u> ELEV/(Z) _____

DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOW COUNT	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	ATTERBERG LIMITS			MATERIAL DESCRIPTION	<200	D ₅₀	USCS GROUP SYMBOL
						LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX				
45			19/25	24.9	99.3				Dark Gray Sandy SILT - Interbedded Fine SAND with Gravel (Wet, Dense to Very Dense)	60	.05	ML
			11/16 /32 (48)			21	17	4				
50			11/22	23.6	103.6				Gray Silty SAND (Wet, Dense)	13	.20	SM
			11/17 /28 (45)						Gray Sandy CLAY - Interbedded Fine SAND Layers (Wet, Hard)			CL
55			11/19	18.8	109.3					55	.06	
			14/18 /17 (33)									
60			25/ 50/57	17.0	117.4				Gray Silty SAND (Wet, Very Dense)	16	.32	SM
			15/27 /30 (57)									
65			10/19	23.5	107.6				Gray Silty SAND with Scattered Gravel (Wet, Very Dense)	37	.15	SM
			22/30 /37 (67)						Gray Sandy SILT (Wet, Very Dense)			ML
70			17/30	18.9	114.4				Gray Silty SAND (Wet, Very Dense)	12	.19	SM
			25/35 /42 (77)						Gray Sandy SILT to Silty SAND (Wet, Very Dense)			SM
75			11/16	22.0	107.4				Gray Silty SAND (Wet, Very Dense)	13	.20	SM
			16/23 /30 (53)									
80	<p>T.D. 77.5' First Water @ 15' Water After Drilling @ 17' Water End of Day @ 17'</p>											
85												



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AGI Geotechnical, Inc. 18555 Sherman Way, Suite A Van Nuys, California 91408 Telephone: (818) 785-6244 Fax: (818) 785-6251

CLIENT	DIDM Development Corp.	PROJECT NAME	Five Story Apartment over Two Level Parking
PROJECT NUMBER	19-3342-00	PROJECT LOCATION	1022-1054 S. La Cienega Blvd.
DATE STARTED	7/17/2009	COMPLETED	7/17/2009
EXCAVATION CONTRACTOR	Chioce Drilling	GROUND ELEVATION	TEST PIT SIZE
EXCAVATION METHOD	8" Hollow stem Auger	GROUND WATER LEVELS:	POINT COORDINATES:
LOGGED BY	W.A.F.	CHECKED BY	J.A.V.
NOTES		AFTER EXCAVATION	ELEV(Z)

DEPTH (ft)	BULK SAMPLE	DRIVE SAMPLE	BLOW COUNT	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	ATTERBERG LIMITS			MATERIAL DESCRIPTION	<200	D ₅₀	USCS GROUP SYMBOL
						LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX				
0												
0-5	X								Artificial Fill: Brown to Dark Brown Sandy CLAY (Moist, Loose to Soft)			CL
5-10			14/17 4/6/8 (14)	13.8	117.2				Natural Ground: Brown Sandy CLAY with Scattered Gravel (Moist, Stiff)			CL
10-15			4/10 2/4/8 (10)	14.6	105.1				Brown Sandy CLAY (Moist, Stiff)			CL
15-20			15/17 2/3/4 (7)	12.6	108.3	27	19	8	Gray Sandy Clay (Moist, Firm)	65	.05	CL
20-25	X		12/20	17.8	109.8				Gray Sandy CLAY (Very Moist, Stiff) (Strong Gasoline Odor)			
25-30			6/11 /22 (33)						Gray Medium to Coarse SAND - Interbedded SILT Layers (Wet, Dense) (Strong Gasoline Odor)			SM
30-35			N/R 14/24 5/10 /22 (32)									
35-40			18/28 15/22 /28 (50)	19.6	103.9				Brown Silty Coarse SAND with Gravel (Wet, Very Dense)			SM
40-45			N/R 16/26 18/24 /34 (58)						Gray Silty SAND to Sandy SILT (Wet, Very Dense)			ML
45-50			11/17 11/12 /14 (26)	22.2	101.9							
50-55			11/15	25.8	99.2				Gray Silty CLAY - Interbedded Fine SAND (Wet, Very Stiff)	81	.04	CL

Borings Performed By:
Geotechnologies
(2012)

BORING LOG NUMBER 1

Assisted Living V, LLC

Date: 06/23/12

File No. 20358

Method: 5-inch diameter Mud Rotary

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				0 --		Surface Conditions: Bare Ground
				-		FILL: Sandy Silt, dark brown, moist, stiff
				1 --		
				-		
2.5	24	11.9	122.3	2 --		
				-		
				3 --		Sandy Clay, dark brown, moist, stiff
				-		
				4 --		
				-		
5	14	15.0	SPT	5 --	SM	ALLUVIUM: Silty Sand, grayish brown, moist to very moist, medium dense, fine to medium grained
				-		
				6 --		
				-		
7.5	52	9.8	123.6	7 --		
				-		
				8 --		brown, moist, dense, some gravel
				-		
				9 --		
				-		
10	7	31.4	SPT	10 --	CL	Silty Clay, medium brown, moist to very moist, firm
				-		
				11 --		
				-		
12.5	17	25.4	101.1	12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	4	46.6	SPT	15 --	CH	Clay, gray and brown, very moist, firm
				-		
				16 --		
				-		
				17 --		
				-		
17.5	10	40.7	88.7	18 --		
				-		
				19 --		
				-		
				20 --		
				-		
20	28	22.1	SPT	20 --	CL	Silty Clay, grayish brown, moist, firm to stiff
				-		
				21 --		
				-		
				22 --		
				-		
22.5	26	29.0	95.0	23 --		
				-		
				24 --		
				-		
				25 --		
25	23	25.7	SPT	25 --	ML	Clayey to Sandy Silt, grayish brown, moist, stiff
				-		

BORING LOG NUMBER 1

Assisted Living V, LLC

File No. 20358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
27.5	54	30.0	93.7	28 --		
				-		
				29 --		
				-		
30	20	26.1	SPT	30 --		
				-	CL	Silty Clay, gray, moist, stiff
				31 --		
				-		
				32 --		
				-		
32.5	38	27.8	98.0	33 --		
				-		
				34 --		
				-		
35	13	36.7	SPT	35 --		
				-		
				36 --		dark gray, slightly more moist
				-		
				37 --		
				-	SM	Silty Sand, gray to dark gray, wet, dense, fine grained
37.5	36	19.8	109.8	38 --		
				-		
				39 --		
				-		
40	44	26.9	SPT	40 --		
				-		
				41 --		
				-		
				42 --		
				-		
42.5	59	26.8	100.6	43 --		
				-		
				44 --		
				-		
45	14	30.8	SPT	45 --		
				-	CL	Silty Clay, gray to dark gray, moist, stiff
				46 --		
				-		
				47 --		
				-		
47.5	57	25.7	101.9	48 --		
				-		
				49 --		
				-		
50	23	23.2	SPT	50 --		
				-	CL	Silty to Sandy Clay, gray to dark gray, moist, stiff

BORING LOG NUMBER 1

Assisted Living V, LLC

File No. 20358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
52.5	70	16.5	116.3	53 --		slightly less moist
				54 --		
				55 --		
55	20	37.7	SPT	56 --	CH	Clay, dark gray, moist to very moist, firm
				57 --		
57.5	52 50/5"	18.7	112.3	58 --	ML	Clayey to Sandy Silt, gray and dark gray, moist, stiff
				59 --		
				60 --		
60	47	30.6	SPT	61 --		very moist to wet
				62 --		
62.5	46 50/5"	18.6	110.1	63 --	SM/SP	Silty Sand to Sand, dark gray, wet, very dense, fine grained
				64 --		
				65 --		
65	60	22.7	SPT	66 --		
				67 --		
67.5	35 50/5"	21.0	104.9	68 --		
				69 --		
				70 --	ML	Sandy Silt, dark gray, moist, firm to stiff
				71 --		Total depth: 70 feet
				72 --		Water at 18 feet
				73 --		Fill to 5 feet
				74 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual
				75 --		Used 5-inch diameter Mud Rotary Drill Rig
						140-lb. Automatic Hammer, 30-inch drop
						Modified California Sampler used unless otherwise noted
						SPT=Standard Penetration Test

BORING LOG NUMBER 2

Assisted Living V, LLC

Date: 06/23/12

File No. 20358

Method: 5-inch diameter Mud Rotary

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				0 --		FILL: Sandy Silt, dark brown, moist, stiff
				1 --		
				2 --		
2.5	37	10.9	129.1	3 --		
				4 --	ML	ALLUVIUM: Sandy Clay, dark brown, slightly moist, stiff
				5 --		
5	23	9.6	126.8	6 --	SM	Silty Sand, brown, moist, medium dense, fine to medium grained, some gravel
				7 --		
7.5	18	30.3	94.8	8 --	CL	Silty Clay, brown, moist, firm
				9 --	ML	Sandy Silt, brown, moist, firm
				10 --		
10	17	10.0	108.5	11 --	SM/SP	Silty Sand to Sand, brown, moist to very moist, medium dense, fine to medium grained
				12 --		
				13 --		
				14 --		
15	22	22.9	105.1	15 --		
				16 --	SM	Silty Sand, brown, moist to wet, medium dense, fine grained
				17 --		
				18 --		
				19 --		
20	23	25.1	102.1	20 --		
				21 --	CL	Silty Clay, grayish brown, moist, stiff
				22 --		
				23 --		
				24 --		
25	48	30.3	94.8	25 --		
					ML	Clayey to Sandy Silt, grayish brown, moist, stiff

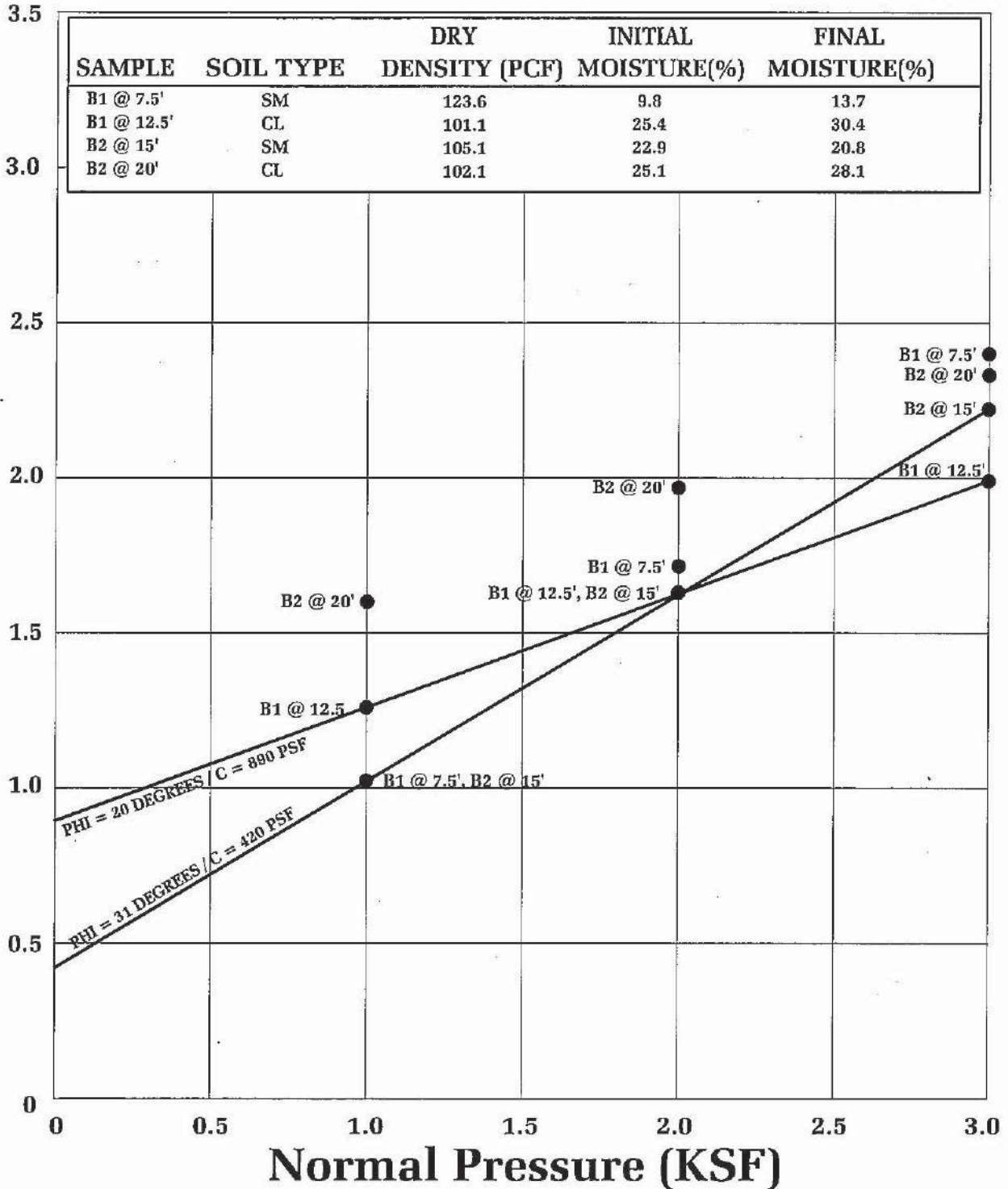
BORING LOG NUMBER 2

Assisted Living V, LLC

File No. 20358

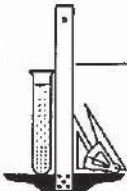
km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
30	46	24.4	103.3	-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
				30 --		
35	63	14.6	120.6	-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
				35 --		
40	49 50/5"	25.8	100.1	-		
				36 --	SM	Silty Sand, gray to dark gray, moist, dense, fine grained
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --	ML	Sandy Silt, dark gray, moist, stiff
-						
41 --			Total depth: 40 feet			
-			Water at 17 feet			
42 --			Fill to 3 feet			
-						
43 --						
-						
44 --						
-						
45 --						
-						
46 --						
-						
47 --						
-						
48 --						
-						
49 --						
-						
50 --						
-						



● Direct Shear, Saturated

SHEAR TEST DIAGRAM

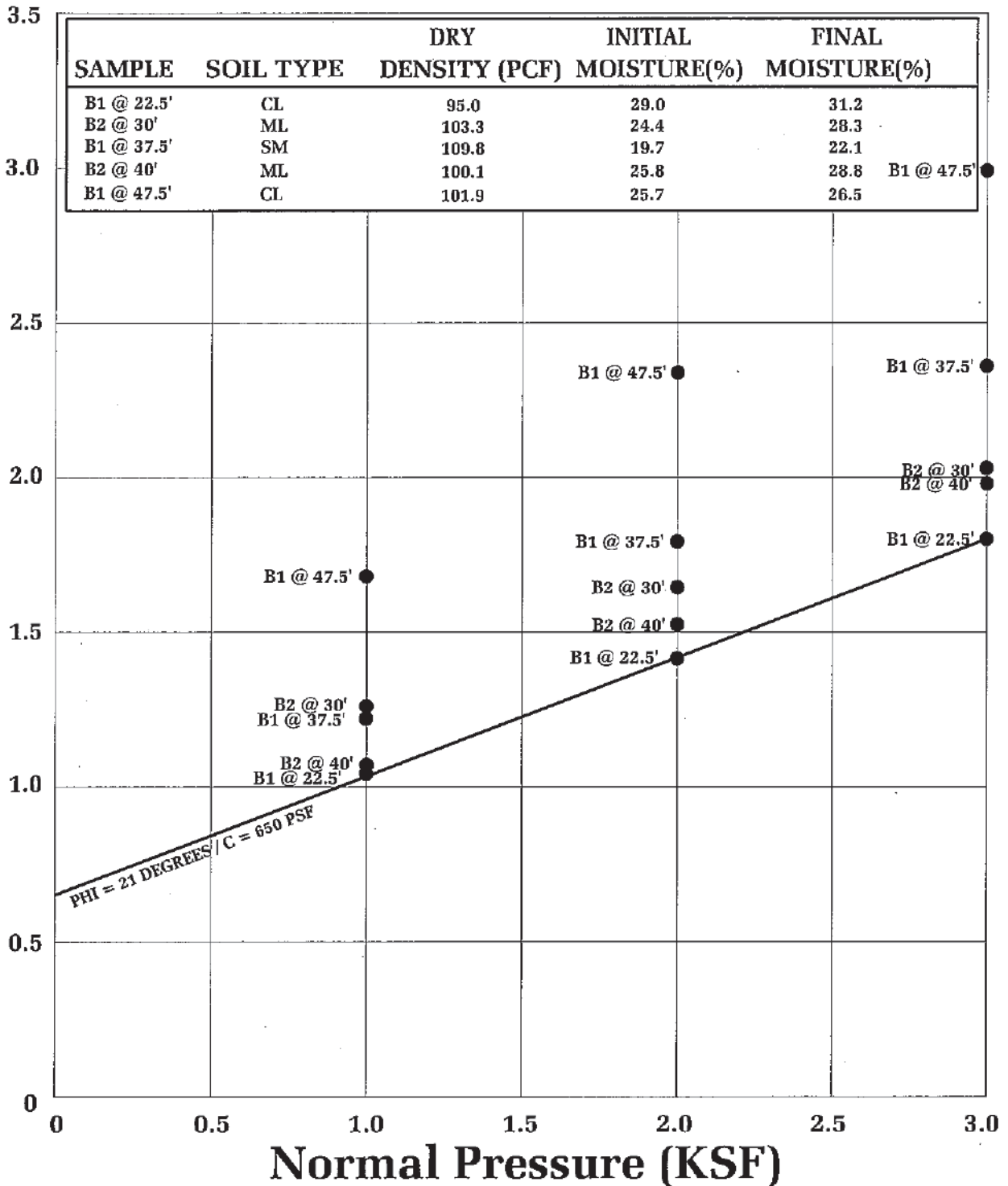


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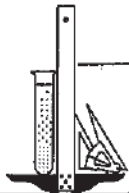
FILE NO. 20358

PLATE: B-1



● Direct Shear, Saturated

SHEAR TEST DIAGRAM



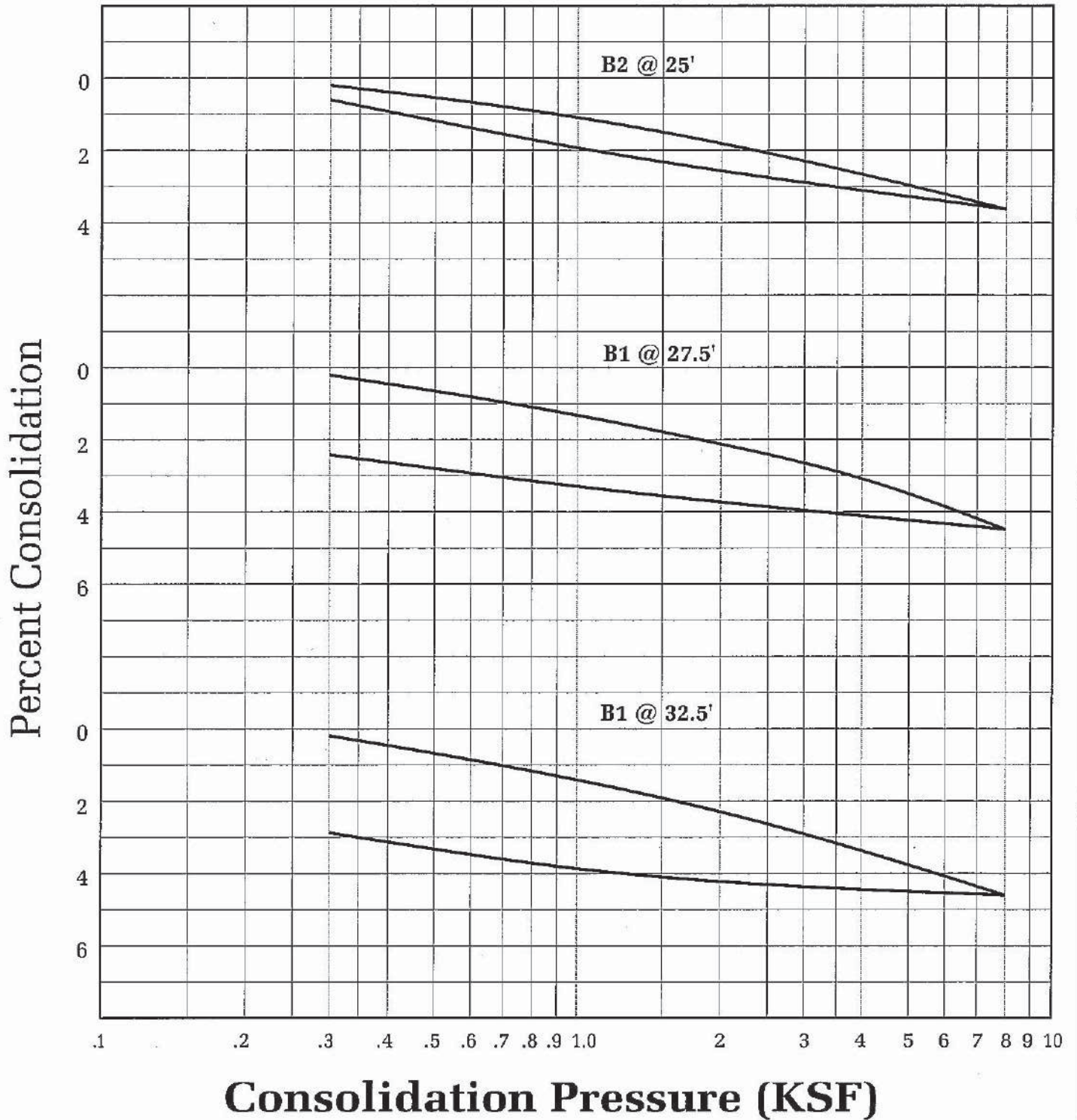
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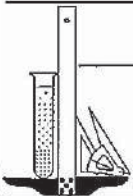
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PLATE: B-2

WATER ADDED AT 2 KSF



CONSOLIDATION TEST



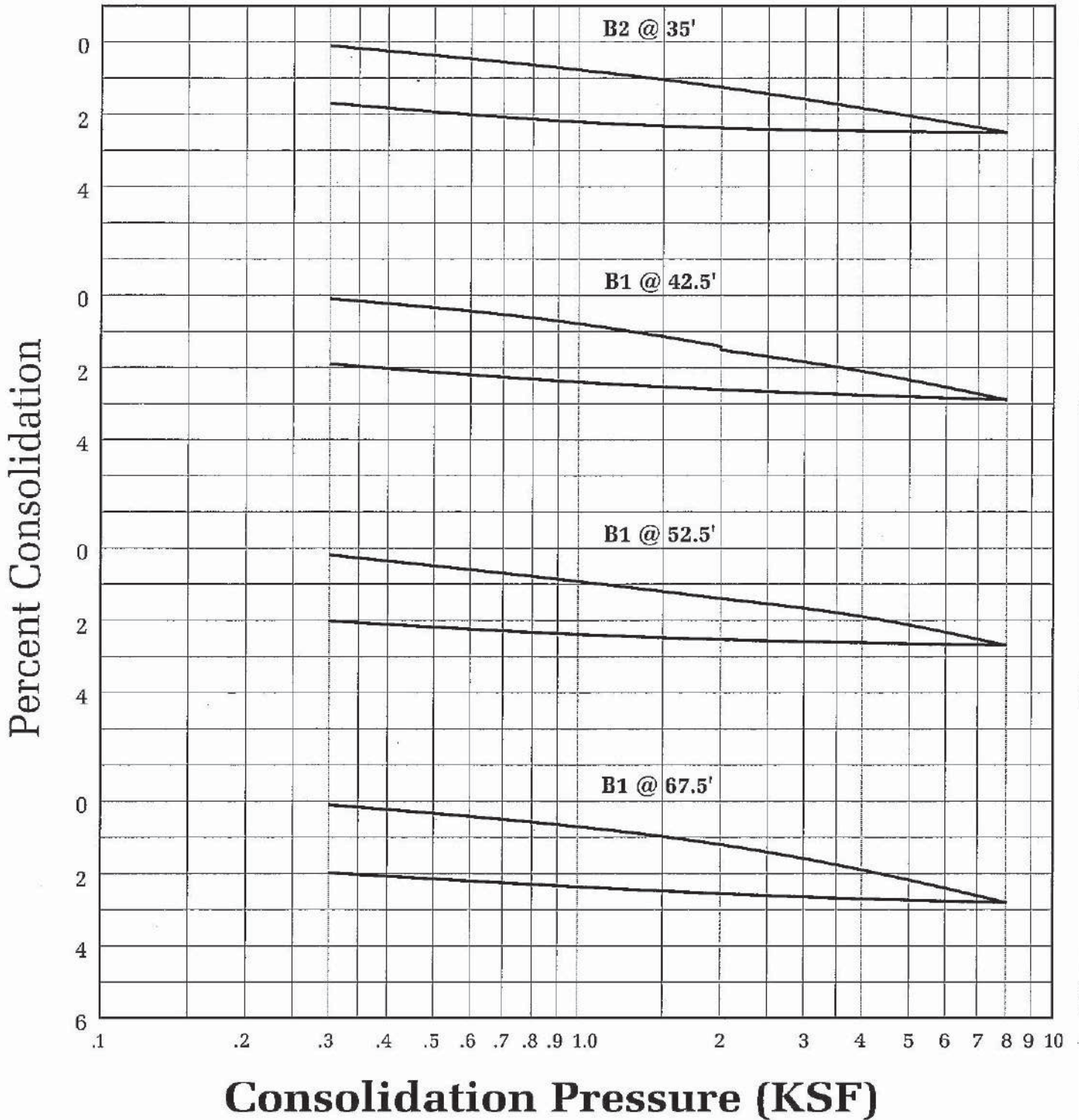
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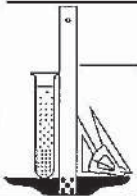
FILE NO. 20358

PLATE: C-1

WATER ADDED AT 2 KSF



CONSOLIDATION TEST



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PLATE: C-2

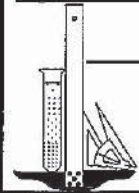
ASTM D 4829

SAMPLE	B2 @ 2.5'	B2 @ 25'	B1 @ 27.5'
SOIL TYPE:	ML	ML	ML
EXPANSION INDEX UBC STANDARD 18-2	73	19	5
EXPANSION CHARACTER	<u>MODERATE</u>	<u>VERY LOW</u>	<u>VERY LOW</u>

SULFATE CONTENT

SAMPLE	B2 @ 2.5'	B2 @ 25'	B1 @ 27.5'
SULFATE CONTENT: (percentage by weight)	< 0.10%	< 0.10%	< 0.10%

COMPACTION/EXPANSION/SULFATE DATA SHEET



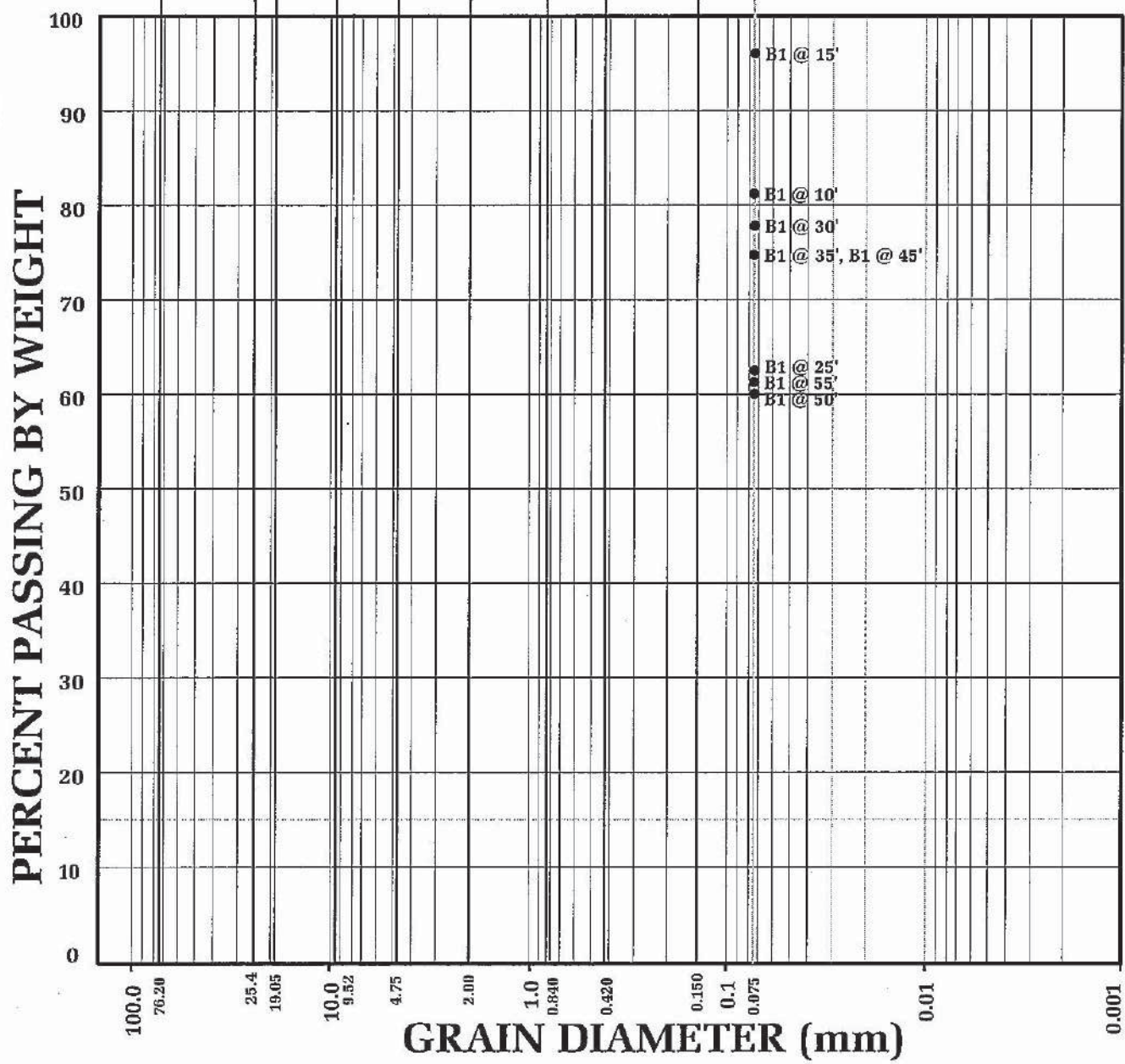
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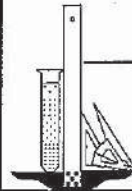
FILE NO. 20358

PLATE: D

GRAVEL	SAND		SILT	CLAY
	MEDIUM TO COARSE	FINE		
U.S. Standard Sieve Sizes				
	3 in.	1 in. 3/4 in.	3/8 in.	NO. 4
				NO. 10
				NO. 20
				NO. 40
				NO. 100
				NO. 200



GRAIN SIZE DISTRIBUTION



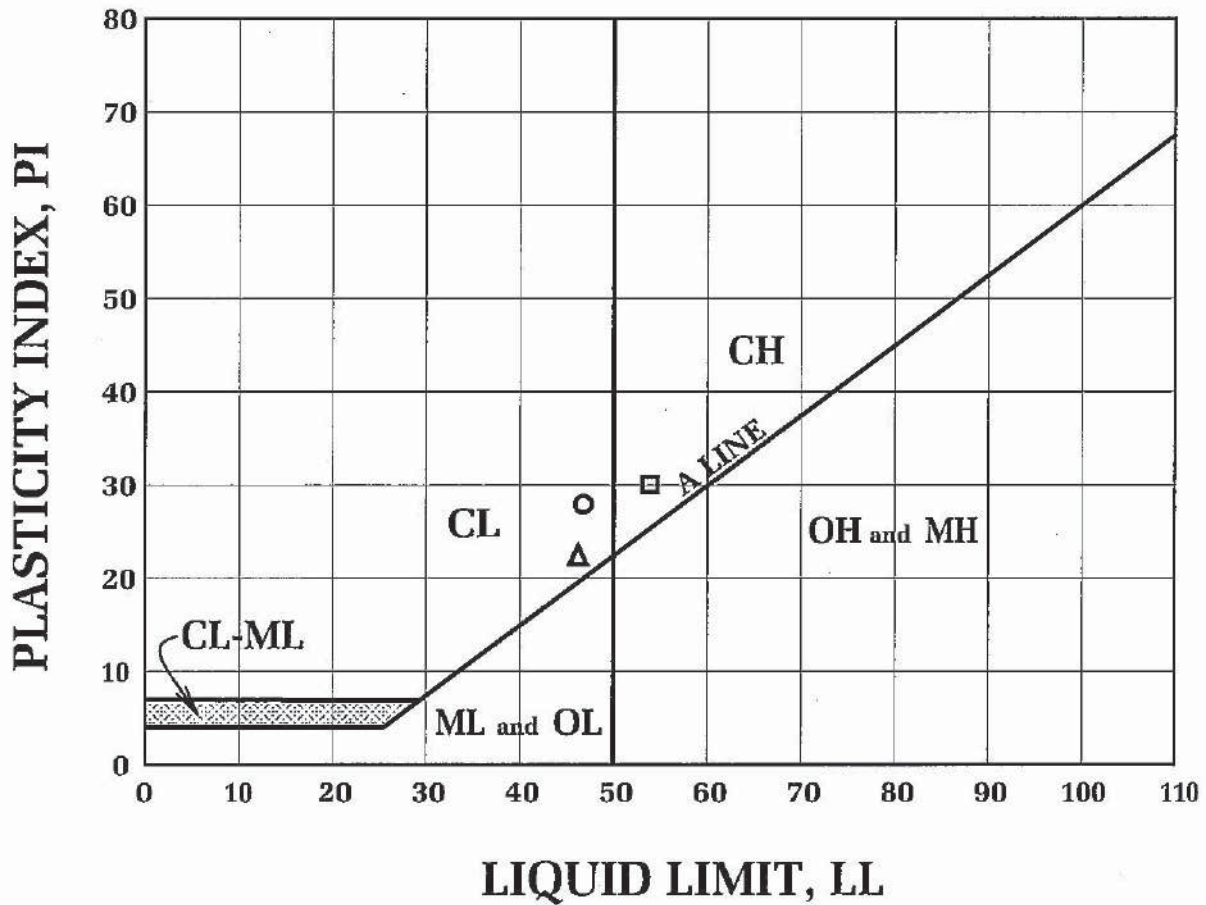
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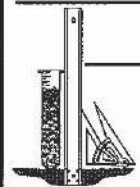
PLATE: E

ASTM D4318



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B1	10	○	47	19	28	CL
B1	15	□	54	24	30	CH
B1	35	△	46	24	22	CL

ATTERBERG LIMITS DETERMINATION



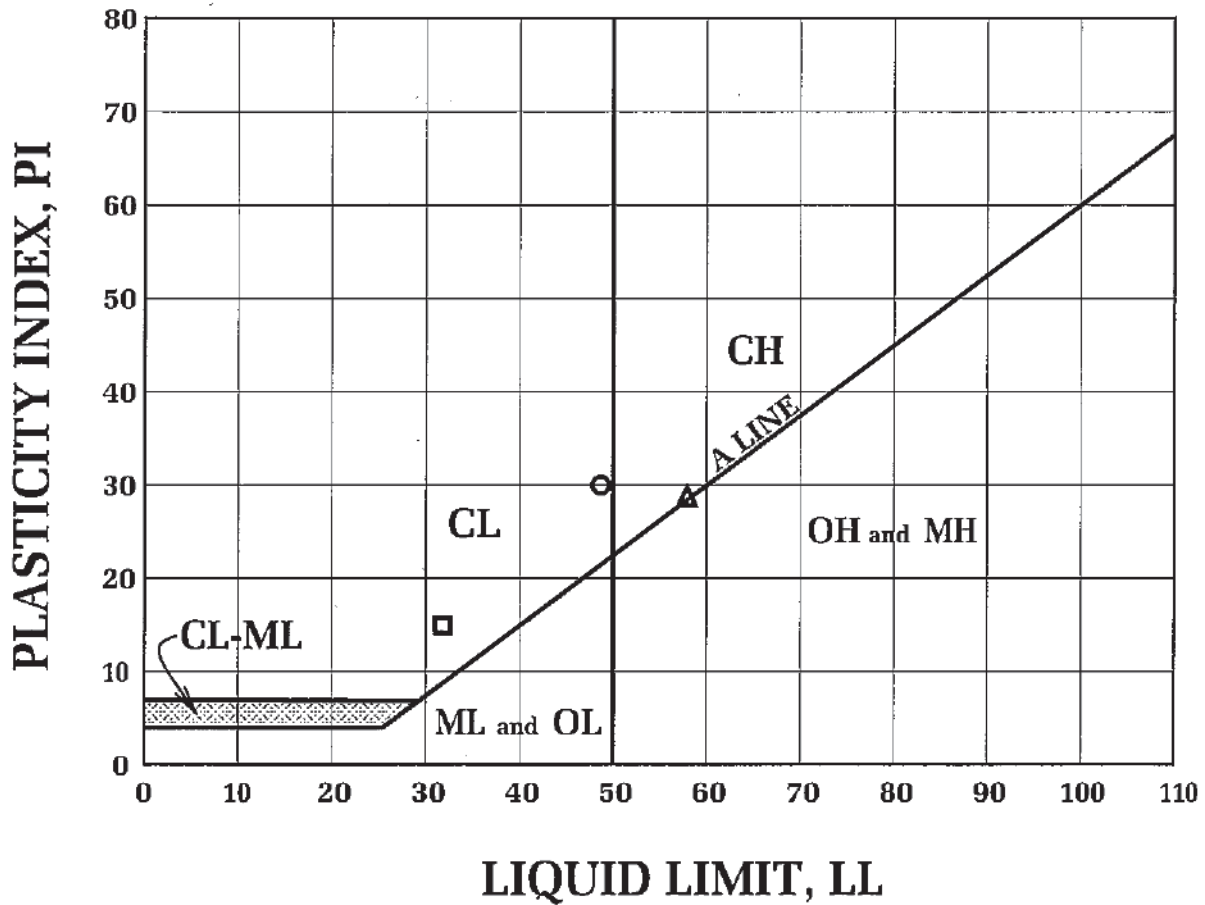
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FILE NO. 20358

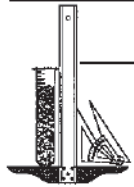
PLATE: F-1

ASTM D4318



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B1	45	○	49	19	30	CL
B1	50	□	32	17	15	CL
B1	55	△	58	30	28	CH

ATTERBERG LIMITS DETERMINATION



Geotechnologies, Inc.
Consulting Geotechnical Engineers

ASSISTED LIVING V, LLC

FILE NO. 20358

PLATE: F-2

APPENDIX B

CURRENT BORING LOGS

B.1 CURRENT BORING LOGS

The current drilling was performed by GeoPentech over the course of two days on August 26-27, 2021 (Borings GP-1 and GP-2), four days on October 26-29, 2021 (Boring GP-3), and two days on February 17-18, 2022 (GP-4). The explorations consisted of advancing four borings: GP-1 to a depth of approximately 50.5 ft, GP-2 to approximately 100.5 ft, GP-3 to approximately 200 ft, and GP-4 to approximately 176.5 ft, below the ground surface. The approximate locations of the borings are indicated on Figure 2 in the main report. The borings were drilled using a combination of an 8-inch diameter hollow stem auger and a 3.875-inch diameter mud rotary drill bit. The drilling was subcontracted to BC2 Environmental, who provided all drilling equipment, crew, and supplies. The work was performed under the supervision of an engineer or a geologist who monitored the drilling operations and prepared a field record of soils observed and drilling conditions.

During drilling, soil samples were obtained at approximate intervals ranging between 2.5 and 5-foot using a Standard Penetration Test (SPT) sampler, a Modified California (CA) sampler, or a Pitcher Barrel sampler. SPT and CA samples were taken by driving a sampler approximately 18 inches into the soil at the bottom of the boring using a 140-pound hammer falling approximately 30 inches. The truck mounted CME 75 rig used by BC2 Environmental utilized an automatic-trip hammer.

The SPT sampler cutting shoe and barrel have nominal inside diameters of 1.375 and 1.50 inches, respectively, and a nominal outside diameter of 2.00 inches. The barrel had no space for internal liners which were not used. The SPT samples were placed in plastic bags, labeled, and sealed. The CA sampler cutting shoe and barrel have nominal inside diameters of 2.38 and 2.50 inches, respectively, and a nominal outside diameter of 3 inches. Nominal 6-inch long, 2.4-inch diameter brass tubes or alternatively assemblies of 1-inch long, 2.4-inch diameter brass rings combined to fill the sampler were used to line the barrel. Plastic end caps were placed on the CA tubes to help preserve the moisture content of the samples. Bulk soil samples were also obtained at certain depths in selected boreholes. Upon completion of drilling, logging, and sampling, all borings were backfilled with cement-bentonite and patched at the surface with soil.

After recovering the sample, the engineer or geologist noted the depth interval, recorded a description of the recovered material onto a field log, and sealed and labeled the sample for transport to the laboratory. The soil descriptions noted on the field logs were visually classified in accordance with the Unified Soil Classification System. The results of the borehole drilling and logging effort are provided on the borehole logs and on a key to the logs of boreholes.

Project: 1056 La Cienega
 Project Location: 1056 La Cienega
 Project Number: 21086A

Key to Log of Boring

Sheet 1 of 1

Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
1	2	3	4	5	6	7	8	9	10	11

COLUMN DESCRIPTIONS

- | | |
|--|--|
| <p>1 Elevation: Elevation in feet referenced to mean sea level (MSL).</p> <p>2 Depth: Depth in feet below the ground surface.</p> <p>3 Sample Type: Type of soil sample collected at depth interval shown; sampler symbols are explained below.</p> <p>4 Sample Number: Sample identification number.</p> <p>5 Sampling Resistance: Number of blows required to advance driven sampler 6 inches, or distance noted, using the drive weight listed in hammer data. Hydraulic down-pressure may be recorded for pushed samplers.</p> <p>6 Sample Recovery: Amount of sample recovered from sampling interval; given as inches of sample recovered or ratio of sample length to drive length (expressed as a percentage, %)</p> | <p>7 Graphic Log: Graphic depiction of subsurface material encountered; typical symbols are explained below.</p> <p>8 Material Description: Description of material encountered; may include density/consistency (from field assessments), moisture, color (Munsell code), and grain size.</p> <p>9 Water Content: Water content of sample, as percentage of dry weight of soil, measured in lab according to ASTM D2216.</p> <p>10 Dry Unit Weight: The weight of soil solids per cubic foot of total volume of soil mass, measured according to ASTM D2937.</p> <p>11 Remarks and Other Tests: Comments and observations regarding drilling or sampling made by driller or field personnel. Other lab tests are indicated using abbreviations explained below.</p> |
|--|--|

TYPICAL MATERIAL GRAPHIC SYMBOLS

Poorly-graded SAND (SP)	POORLY GRADED SAND W/ SILT (SP-SM)	WELL GRADED SAND W/ SILT (SW-SM)	Silty SAND (SM)
Clayey SAND (SC)	SILT (ML)	LEAN CLAY (CL)	FAT CLAY (CH)

TYPICAL SAMPLER GRAPHIC SYMBOLS

Grab Sample	Modified California
Standard Penetration Test	Pitcher Barrel Sample

OTHER LABORATORY TEST ABBREVIATIONS

- CONSOL1-D** Consolidation testing (ASTM D2435)
CORR Corrosion testing (DOT CA test methods 643, 417, 422)
DS Consolidated drained direct shear test (ASTM D3080)
FC Fines Content wash on #200 sieve (ASTM D1140)
LL Liquid Limit from Atterberg Limits test (ASTM D4318)
PI Plasticity Index; NP indicates non-plastic determination
G=XX% Percentage (%) Gravel
S=XX% Percentage (%) Sand
F=XX% Percentage (%) Fines

OTHER GRAPHIC SYMBOLS

- Contact between strata
- - - Inferred contact between strata or gradational change
- ▼ Change within material properties within a stratum

Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; field descriptions have been modified to reflect lab test results. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times. Datum used is WGS84.

Project: 1056 La Cienega
 Project Location: 1056 La Cienega
 Project Number: 21086A

Log of GP-1

Sheet 1 of 4

Date(s) Drilled	08/26 - 8/27/2021	Logged By	M. Eslami	Checked By	R. Hadidi
Drilling Method	Hollow Stem Auger & Mud Rotary	Drill Bit Size/Type	8" HSA / 3-7/8" MR drill bit	Total Depth of Borehole	100.5 feet
Drill Rig Type	CME 75	Drilling Contractor	BC2 Environmental	Approximate Surface Elevation	123 ft
Groundwater Level(s)	8' bgs	Sampling Method	Bulk, SPT, MC	Hammer Data	Automatic Trip Hammer 140-lbs/30" drop
Borehole Location	34.058463°, -118.375910°	Borehole Completion	Borehole backfilled with cement bentonite slurry using tremie pipe from bottom of hole to surface.		

Elevation, feet	Depth, feet	SAMPLES			Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"					
0			Bulk-1						[Artificial Fill (Af)] Silty SAND (SM), moist, dark brown Start with HSA
120			1	4 4 6	18"/18"				[Alluvium (Qa)] Well-Graded SAND with silt (SW-SM), loose, moist, dark brown, fine to coarse grained, non-plastic; no HCl reaction 0-5' CORR
5			2a 2b	4 9 9	18"/18"		104.0	5.0	becomes medium dense, few gravel 2a FC = 7.6% 2b G=14%, S=79%, F=7%
115			3a 3b	8 12 19	18"/18"				becomes dense, wet, brown Perched water @ 8'
10			4a 4b	7 14 15	18"/18"			13.8	Poorly-graded SAND with silt (SP-SM), medium dense, wet, brown, non-plastic, trace gravel; no HCl reaction @ 10' bgs: Switch to Mud Rotary 4a G=5%, S=89%, F=6%
110			5	6 11 13	12"/18"				same as above
15			6a 6b	1 5 7	14"/18"				Poorly-graded SAND (SP), medium dense, wet, brown, angular grains, trace gravel 6a G=4%, S=93%, F=3% DS
105			7	5 8 10	12"/18"			18.5	Silty SAND (SM), medium dense, wet, brown, angular grains, non-plastic LL = NP PI = NP FC = 12.2%
100			--	1 4 10	NR				
25			8	7 6 12	18"/18"				Sandy CLAY (CL), very stiff, wet, grayish brown, medium plasticity
95			9a	10 26	18"/18"		108.1	17.2	Clayey SAND (SC), dense, wet, grayish brown, medium plasticity 9a LL = 47

Report: GP SOIL BA LOG; File: 21086A 1056 LA CIENEGA.GPJ; 3/30/2022

Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
30		9b	42			Lean CLAY (CL), very stiff, moist, grayish brown, medium plasticity			PI = 25 FC = 34.9% CONSOL 9b DS	
90										
35		10	6 12 20	18"/18"		becomes hard, wet, dark gray, includes fine grained sand; no HCl reaction	26.4		LL = 30 PI = 12	
85										
40		11a 11b	18 22 31	18"/18"		Poorly-Graded SAND with silt (SP-SM), dense, moist, dark gray, non-plastic	26.1			
80										
45		12	3 4 8	18"/18"		Lean CLAY (CL), stiff, moist, dark gray, low plasticity; no HCl reaction	31.8		LL = 43 PI = 31 FC = 84.1%	
75										
50		13a 13b	14 18 31	18"/18"		becomes hard, medium plasticity	98.7	27.9	13a LL = 28 PI = 13 FC = 88.8% CONSOL	
70										
55		14	3 4 7	18"/18"		becomes stiff, sandy			LL = 35 PI = 19 FC = 66.8%	
65										
60		15a 15b	12 16 28	18"/18"		becomes hard, slightly plastic	104.6	25.6	15a LL = 44 PI = 28 FC = 70.3% CONSOL	
60										
65		16	21 22	18"/18"		Silty SAND (SM), dense, wet, dark gray, fine grained; no HCl reaction		26.6	FC = 44.7%	

Report: GP SOIL BA LOG; File: 21086A 1056 LA CINEGA.GPJ; 3/30/2022

Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
65				23						
55										
70		17	8"/18"	10 20 30		Same as above				
50										
75		18	18"/18"	19 19 23		slight odor				
45										
80		19	18"/18"	7 9 20		Fat CLAY (CH), very stiff, moist, dark gray, high plasticity		38.1	LL = 51 PI = 25 FC = 96.2%	
40										
85		20a 20b	18"/18"	22 35 48		Sandy SILT (ML), hard, moist, dark gray, non-plastic, fine grained; no HCl reaction	101.2	27.4	20a FC = 65.9%	
35										
90		--		25 26 26	NR				Driller mentioned hose in hard material	
30										
95		21a 21b	11"/18"	46 50/5"		Silty SAND (SM), very dense, moist, dark gray, fine grained, non-plastic; no HCl reaction			21a FC = 49.2%	
25										
100		22	18"/18"	6 10		Fat CLAY (CH), very stiff, moist, dark gray, high plasticity			LL = 64 PI = 45	

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Project: 1056 La Cienega
 Project Location: 1056 La Cienega
 Project Number: 21086A

Log of GP-1

Sheet 4 of 4

Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS	
		Type	Number	Blows Per 6"	Recovery						
100		█		13		███				FC = 95.1%	
20										<p>Total depth: 100.5' bgs</p> <p>Borehole backfilled with cement bentonite slurry using tremie pipe from bottom of hole to surface.</p> <p>Perched groundwater observed at 8' bgs.</p>	
105											
15											
110											
10											
115											
5											
120											
0											
125											
-5											
130											
-10											
135											

Report: GP SOIL BA LOG; File: 21086A 1056 LA CINEGA.GPJ; 3/30/2022

Project: 1056 La Cienega
 Project Location: 1056 La Cienega
 Project Number: 21086A

Log of GP-2

Sheet 1 of 2

Date(s) Drilled	08/27/2021	Logged By	M. Eslami	Checked By	R. Hadidi
Drilling Method	Hollow Stem Auger & Mud Rotary	Drill Bit Size/Type	8" HSA / 3-7/8" MR drill bit	Total Depth of Borehole	50.5 feet
Drill Rig Type	CME 75	Drilling Contractor	BC2 Environmental	Approximate Surface Elevation	122 ft
Groundwater Level(s)	Not Observed	Sampling Method	Bulk, SPT, MC	Hammer Data	Automatic Trip Hammer 140-lbs/30" drop
Borehole Location	34.057602°, -118.375837°	Borehole Completion	Borehole backfilled with cement bentonite slurry using tremie pipe from bottom of hole to surface.		

Elevation, feet	Depth, feet	SAMPLES			Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS	
		Type	Number	Blows Per 6"						Recovery
0			Bulk-1						[Artificial Fill (Af)] Sandy CLAY (CL), moist, dark brown	Start with HSA
120			1a 1b	10 11 16	18"/18"		117.7	11.5	Sandy Fat CLAY (CH), very stiff, brown, moist, brown, highly plastic, sandy; no HCl reaction	1a LL = 50 PI = 37 G=1%, S=45%, F=54%
5			2	8 6 6	18"/18"				Sandy CLAY (CL), stiff, brown, moist, brown, slightly plastic, sandy; no HCl reaction	LL = 41 PI = 27
115			3a 3b	2 7 11	18"/18"		108.6	17.9	[Alluvium (Qa)] Lean CLAY (CL), stiff, moist, brown, medium plasticity, some fine grained sand; no HCl reaction	3a FC = 57.9% DS 3b G=0%, S=44%, F=56%
10			4	1 5 11	18"/18"				becomes very stiff	@ 10' bgs: Switch to Mud Rotary
110			5a 5b	7 13 16	18"/18"		94.1	28.3	Sandy SILT (ML), very stiff, moist, brown, slightly plastic, very fine grained; no HCl reaction becomes fine to coarse grained, angular, non-plastic, trace gravel	5a FC = 67.7%
15			6	7 13 16	4"/18"					
105			7a 7b	7 12 15	18"/18"		109.5	11.0	Clayey SAND (SC), medium dense, moist, brown, medium plasticity; no HCl reaction	7a LL = 53 PI = 37 G=14%, S=61%, F=25%
20			8	3 3 9	18"/18"				Lean CLAY (CL), stiff, moist, brown, low plasticity; no HCl reaction	
100			9a 9b	3 9 17	18"/18"		92.1	31.3	becomes very stiff	9a LL = 41 PI = 16 FC = 97.2% DS CONSOL
95										
30			10	6 10	12"/18"				Same as above	LL = 34 PI = 12

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Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
30				15						
90										
35		11a 11b		17 20 22	18"/18"	Silty SAND(SM), dense, wet, brown, very fine grained, non-plastic; no HCl reaction	106.0	21.8	11a FC = 43.4% 11b LL = NP PI = NP FC = 27%	
85										
40		12		5 12 13	18"/18"	Sandy SILT (ML), very stiff, moist, dark gray, slightly plastic, fine grained; no HCl reaction, some odor				
80										
45		13a 13b		5 8 12	18"/18"	Lean CLAY (CL), very stiff, moist, dark gray, medium plasticity; no HCl reaction	92.0	31.1	13a LL = 41 PI = 23 FC = 95.6% CONSOL	
75										
50		14a 14b		20 34 47	18"/18"	Sandy SILT (ML), hard, moist, dark gray, non-plastic, some gravel; strong HCl reaction			14a FC = 55.1%	
70						Total depth: 50.5' bgs Borehole backfilled with cement bentonite slurry using tremie pipe from bottom of hole to surface. Groundwater not observed during drilling; based on monitoring wells on site GWT @ 20' bgs at location GP-2				
55										
65										
60										
60										
65										

Report: GP SOIL BA LOG; File: 21086A 1056 LA CINEGA.GPJ; 3/30/2022

Project: 1056 La Cienega
 Project Location: 1056 La Cienega
 Project Number: 21086A

Log of GP-3

Sheet 1 of 6

Date(s) Drilled	10/26 - 10/29/2021	Logged By	M. Eslami	Checked By	R. Hadidi
Drilling Method	Hollow Stem Auger & Mud Rotary	Drill Bit Size/Type	5" MR drill bit	Total Depth of Borehole	199.3 feet
Drill Rig Type	CME 75	Drilling Contractor	BC2 Environmental	Approximate Surface Elevation	125 ft
Groundwater Level(s)	Obscured by drilling fluid	Sampling Method	Pitcher Barrel, SPT, MC	Hammer Data	Automatic Trip Hammer 140-lbs/30" drop
Borehole Location	34.057623°, -118.375849°	Borehole Completion	Borehole backfilled with cement slurry using tremie pipe from bottom of hole to surface.		

Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
125	0									
			Bulk-1				[Artificial Fill (Af)] Silty SAND (SM), moist, dark brown			HSA 0-10 ft depth
120	5		1a 1b	5 9 15			[Alluvium (Qa)] Well graded SAND with Silt (SW-SM), medium dense, damp, dark brown, slightly plastic, slight HCL rxn			
115	10		2a 2b	2 4 6			Sandy CLAY to Clayey SAND (SC/CL), loose/stiff, moist, medium plasticity			Switch to mud rotary 2a FC = 94.7% LL=55, PI=38
110	15		3a 3b	2 2 4			Silty SAND (SM), poorly graded, loose, brown, moist, medium plasticity, no HCL rxn			3a FC = 83.3%
105	20		4a 4b	1 8 11			Sandy CLAY (CL), very stiff, dark gray, wet, medium plasticity, slight HCL rxn	94.1	22.0	Occasional coarse sand in sample with roots and wood 4a LL=55, PI=36 4b DS
100	25		5a 5b	12 14 16			Sandy SILT to Silty SAND (SM/ML), brown, FeO staining, medium dense, moist, non-plastic, strong rxn to HCL	88.3	30.9	5a FC=96.3% 5b DS
95	30						Silty to Clayey SAND w/ Gravel (SC-SM), wet, dark gray			

Report: GP SOIL BA LOG; File: 21086A 1056 LA CINEGA.GPJ; 3/30/2022

Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
95	30		6						6 G=0%, S=9%, F=91% LL=58, PI=43	
			7a 7b	18 24 28					7a FC=69.9%	
			8a 8b	21 15 19					8a FC=19.7%	
90	35									
			9a 9b	14 19 20				105.5	21.2	9a FC=19.8%
85	40									
			10a 10b	10 16 23						
80	45									
			11					109.1	20.4	11 FC=71.1% LL=37, PI=19 CONSOL
			12a 12b	20 30 45						
75	50									
			13a 13b	6 7 10						
70	55							105.1	21.6	13a FC=55.5% LL=30, PI=18 CONSOL
			14							
			15a 15b	15 25 38						
65	60									
			16a	30 45						
60	65									

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Project: 1056 La Cienega
 Project Location: 1056 La Cienega
 Project Number: 21086A

Log of GP-3

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Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
60	65	☒	16b	50/3"						
55	70	☒	17a 17b	25 45 50/3"						
50	75	☒	18a 18b	28 38 48						
45	80	☒	19a 19b	26 34 44		↓ becomes more fine-grained			19a FC=87.1% LL=NP, PI=NP	
40	85	☒	20a 20b	38 41 50/5"		↓ becomes more coarse grained				
35	90	☒	21a 21b	41 50 44						
30	95	☒		50/4"	NR					
25	100	☒	22a	14 20			Fat CLAY (CH), damp, hard, dark gray			

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Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
25	100	☒	22b	31			90.6	32.3	22a FC=97.7% LL=73, PI=53 CONSOL	
20	105	▨	23				94.2	28.6	23 FC=87.0% LL=51, PI=32 CONSOL	
15	110	☒	24a 24b	25 36 47					24a FC=53.0% LL=42, PI=27	
10	115	☒	25a 25b	27 34 47					25a FC=43.8%	
5	120	☒	26a 26b	28 39 50/4"						
0	125									
-5	130	☒	27a 27b	22 33 43					27a LL=41, PI=27	
-10	135									

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Elevation, feet	Depth, feet	SAMPLES				MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery				
-10	135								
-15	140	28a 28b	18 27 48			Fat CLAY (CH), dark gray, damp, hard, high plasticity, weak HCL rxn	96.9	28.2	28a LL=51, PI=32 CONSOL
-20	145								
-25	150	29	50/3"	NR		Silty SAND, well to poorly graded, wet, dark gray, very dense			Some odor in sample 29 FC=6.7%
-30	155								
-35	160		100/1"	NR					Rig chattering
-40	165								
-45	170		100/1"	NR					Rig chattering

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Project: 1056 La Cienega
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Log of GP-3

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Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
-45	170									
-50	175									
-55	180									
-60	185	▣	30	100/5"	3"	▼ becomes Silty SAND w/ Clay				
-65	190									
-70	195									
-75	200	▣	31	100/4"	4"	same as above				
-80	205					<p>Total depth: 199.3' bgs</p> <p>Borehole backfilled with cement bentonite slurry using tremie pipe from bottom of hole to surface.</p> <p>Groundwater not observed during drilling; obscured by drilling fluid</p>				

Report: GP SOIL BA LOG; File: 21086A 1056 LA CINEGA.GPJ; 3/30/2022



Project: 1056 La Cienega
 Project Location: 1056 La Cienega
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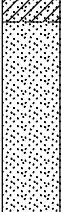
Log of GP-4

Sheet 1 of 6

Date(s) Drilled	2/17 - 2/18/2022	Logged By	W. Erickson	Checked By	R. Hadidi
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8" HSA	Total Depth of Borehole	176.5 feet
Drill Rig Type	CME 75	Drilling Contractor	BC2 Environmental	Approximate Surface Elevation	122 ft
Groundwater Level(s)	20' bgs	Sampling Method	Bulk, SPT, MC	Hammer Data	Automatic Trip Hammer 140-lbs/30" drop
Borehole Location	34.057746°, -118.375923°	Borehole Completion	Borehole backfilled with cement slurry using tremie pipe from bottom of hole to surface.		

Elevation, feet	Depth, feet	SAMPLES			Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"					
0						[Artificial Fill (Af)] Fat CLAY (CH), dry, dark brown, highly plastic, little sand, fine grained			Hand Auger 0-5 ft Start at 8:50 2/17/22
5			1	8 13 12		Clayey SAND (SC), medium dense, dry, dark brown, medium to coarse sand, slightly plastic, slight reaction to HCl			Switch to HSA
10			2	10 8 9		[Alluvium (Qa)] Silty SAND (SM), medium dense, dry, brown, fine to medium sand, nonplastic, no reaction to HCl			
15			3	11 21 30		Well-Graded SAND with Gravel (SW), dense, dry, light brown, fine to coarse sand, gravel to 1", no reaction to HCl			
20			4	6 10 11		Silty SAND with Gravel (SM), medium dense, wet, light brown, fine to coarse sand, nonplastic, gravel to 1", no reaction to HCl			Encountered Groundwater Begin Adding Mud FC=24.1%
25			5	6 7 17		Lean CLAY (CL), very stiff, moist, light brown (mottled), slightly plastic, no reaction to HCl			
30									

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Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
30		⊗	6	9 16 24			becomes sandy, fine grained, slight reaction to HCl	101.8	23.8	LL=33, PI=10 FC=69.9%
90										
35		⊗	7	10 15 24			becomes less sandy, medium plastic			
85										
40		⊗	8	11 19 40			Clayey SAND (SC), dense, wet, dark grey, fine grained, slightly plastic, no reaction to HCl			FC=47.1%
80										
45		⊗	9	4 12 19			Lean CLAY (CH), very stiff, wet, olive grey, highly plastic, no reaction to HCl	102.9	24.2	LL=38, PI=21 FC=90.4% CONSOL
75										
50		⊗	10	14 20 29			same as above			DS
70										
55		⊗	11	12 25 40			Clayey SAND (SC), dense, moist, light grey, fine to coarse sand, slightly plastic, strong reaction to HCl			FC=27.8%
65										
60		⊗	12	13 17 25			Poorly Graded SAND (SP), medium dense, wet, grey, coarse grained, no reaction to HCl			
60										
65										

Report: GP SOIL BA LOG; File: 21086A 1056 LA CINEGA.GPJ; 3/30/2022

Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
65		13	5 21 45			Silty SAND (SM) , dense, wet, grey, medium grained, nonplastic, no reaction to HCl			FC=18.1%	
55										
70		14	14 26 40			Poorly Graded SAND (SP) , dense, wet, grey, fine to medium grained, slight reaction to HCl			DS	
50										
75		15	19 32 50/4"			Clayey SAND (SC) , very dense, wet, grey, fine grained, slightly plastic, no reaction to HCl				
45										
80		16	7 17 26			Fat CLAY (CH) , very stiff, wet, olive grey, highly plastic, no reaction to HCl	93.6	32.4	LL=51, PI=30 FC=85.3% CONSOL	
40										
85		17	19 33 50/3"			Lean CLAY (CL) , very stiff, wet, olive grey, highly plastic, no reaction to HCl				
35										
90		18	4 7 19			Silty SAND (SM) , medium dense, wet, grey, fine-grained, nonplastic, no reaction to HCl				
30										
95		19	17 50/6"			Lean SILT (ML) , hard, wet, grey, nonplastic, no reaction to HCl				
25										
100										

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Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
100		▼	20	6 11 36						
						becomes clayey, low plasticity	94.6	29.3	LL=NP, PI=NP FC=93.0%	
20										
105		▼	21	17 34 35		Clayey SAND (SC), dense, wet, grey, fine-grained, low plasticity, no reaction to HCl				
15										
110		▼	22	9 17 42		Lean SILT (ML), very stiff, wet, dark grey, fine-grained, nonplastic, no reaction to HCl	97.6	28.1	FC=62.9% DS	
10										
115		▼	23	10 20 33		Sandy Lean CLAY (CL), very stiff, wet, grey, medium plasticity, strong reaction to HCl				
5										
120		▼	24	9 28 35		Clayey SAND (SC), very dense, wet, grey, fine to medium grained, low plasticity, strong reaction to HCl	116.9	16.3	LL=36, PI=22 FC=31.1%	
0										
125		▼	25	12 21 40		Sandy Lean CLAY (CL), very stiff, wet, grey, medium plasticity, strong reaction to HCl				
-5										
130		▼	26	9 18 32		Clayey SAND (SC), very dense, wet, grey, fine to medium grained, low plasticity, no reaction to HCl			LL=23, PI=9 FC=32.3%	
-10										
135										

Report: GP SOIL BA LOG; File: 21086A 1056 LA CINEGA.GPJ; 3/30/2022

Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
135		▼	27	20 32 50/5"		▼ same as above				
-15										
140		▼	28	18 31 50/5"		Sandy Lean CLAY (CL), stiff, wet, grey, medium plasticity, no reaction to HCl	92.2	37.0	LL=41, PI=24 FC=94.6%	
-20										
145		▼	29	15 18 26		▼ same as above			Stop at 4:00 2/17/22 Start at 8:00 2/18/22	
-25										
150		▼	30	10 28 50/5"		▼ same as above	94.8	28.5	LL=45, PI=27 FC=58.3%	
-30										
155		▼	31	8 21 50/5"		Fat CLAY (CH), stiff, wet, grey, high plasticity, no reaction to HCl	95.7	30.4	LL=60, PI=45 FC=50.3% CONSOL	
-35										
160		▼	32	23 41 50/4"		Poorly-Graded Silty SAND (SP-SM), very dense, wet, grey, medium grained, nonplastic, no reaction to HCl			FC=7.6%	
-40										
165		▼	33	21 37 50/4"	NR				Driller Notes Possible Gravels	
-45										
170										

Report: GP SOIL BA LOG; File: 21086A 1056 LA CINEGA.GPJ; 3/30/2022

Project: 1056 La Cienega
 Project Location: 1056 La Cienega
 Project Number: 21086A

Log of GP-4

Sheet 6 of 6

Elevation, feet	Depth, feet	SAMPLES				Graphic Log	MATERIAL DESCRIPTION	Dry Unit Weight, pcf	Water Content, %	REMARKS
		Type	Number	Blows Per 6"	Recovery					
170		Hatched	34	25						
				42						50/3"
-50						encountered gravels up to 1"				
175		Hatched	35	4						
				5						5
-55						no gravels encountered			Stop at 4:00 2/18/22	
						<p>Total depth: 176.5' bgs</p> <p>Borehole backfilled with cement bentonite slurry using tremie pipe from bottom of hole to surface.</p> <p>Groundwater observed at 20 ft bgs</p>				
180										
-60										
185										
-65										
190										
-70										
195										
-75										
200										
-80										
205										

Report: GP SOIL BA LOG; File: 21086A 1056 LA CINEGA.GPJ; 3/30/2022



APPENDIX C

CONE PENETRATION TESTING

C.1 CONE PENETRATION TESTING

The Cone Penetration Testing (CPT) was performed by GeoPentech over the course of one day on October 21, 2021. The explorations consisted of advancing three CPTs: CPT-1 to a depth of approximately 91.5 ft, CPT-2 to approximately 74 ft, and CPT-3 to approximately 66 ft, below the ground surface. The approximate locations of the CPTs are indicated on Figure 2 in the main report. The work was subcontracted to ConeTec, who provided all equipment, crew, and supplies.

The following pages contains ConeTec's report and data files.

PRESENTATION OF SITE INVESTIGATION RESULTS

1056 La Cienega Blvd

Prepared for:

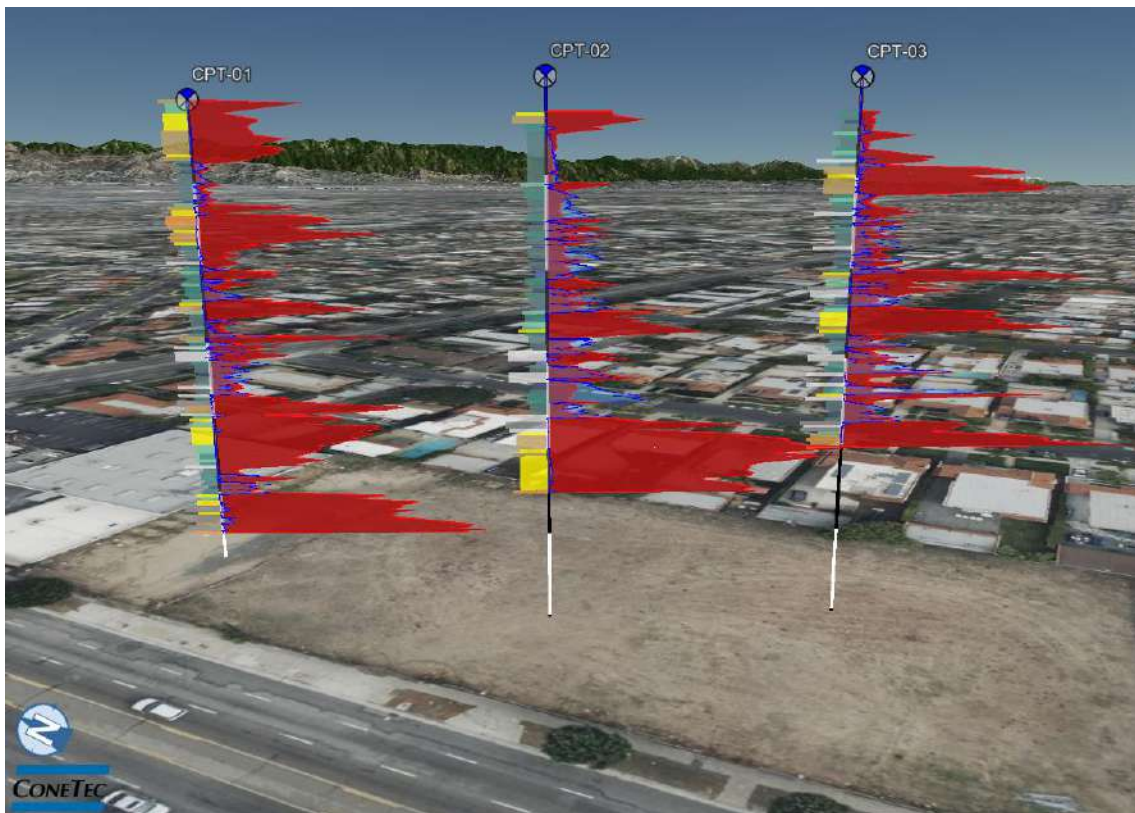
GeoPentech

ConeTec Job No: 21-56-23177

Project Start Date: 21-Oct-2021

Project End Date: 21-Oct-2021

Report Date: 26-Oct-2021



Prepared by:

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Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for GeoPentech of Irvine, California. The program consisted of cone penetration testing (CPTu) at three (3) locations. The assumed phreatic surface used for the calculated parameters is based on the pore pressure dissipation tests performed within or nearest each sounding.

Project Information

Project	
Client	GeoPentech
Project	1056 La Cienega Blvd
ConeTec Project #	21-56-23177

An aerial overview from Google Earth including the CPTu test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C17)	30-ton truck mounted cylinder	CPTu

Coordinates		
Test Type	Collection Method	EPSG Number
CPTu	Consumer grade GPS	32610



Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm ²)	Sleeve Area (cm ²)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
814:T1500F15U35	814	15	225	1500	15	35
Cone 814 was used for all soundings.						

Cone Penetration Test	
Depth reference	Depths are referenced to the existing ground surface at the time of test.
Tip and sleeve data offset	0.1 Meter This has been accounted for in the CPT data files.
Additional Comments	Advanced plots, Normalized plots, Seismic plots, as well as Soil Behavior Type (SBT) Scatter plots have been included in the data release package.

Calculated Geotechnical Parameter Tables	
Additional information	<p>The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2).</p> <p>Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile.</p> <p>Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).</p>

Limitations

This report has been prepared for the exclusive use of GeoPentech (Client) for the project titled “1056 La Cienega Blvd”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec, Inc. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety. www

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm² and 15 cm² tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm² penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm² piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u₂" position ([ASTM Type 2](#)). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current [ASTM D5778](#) standard. ConeTec's calibration criteria also meets or exceeds those of the current [ASTM D5778](#) standard. An illustration of the piezocone penetrometer is presented in [Figure CPTu](#).

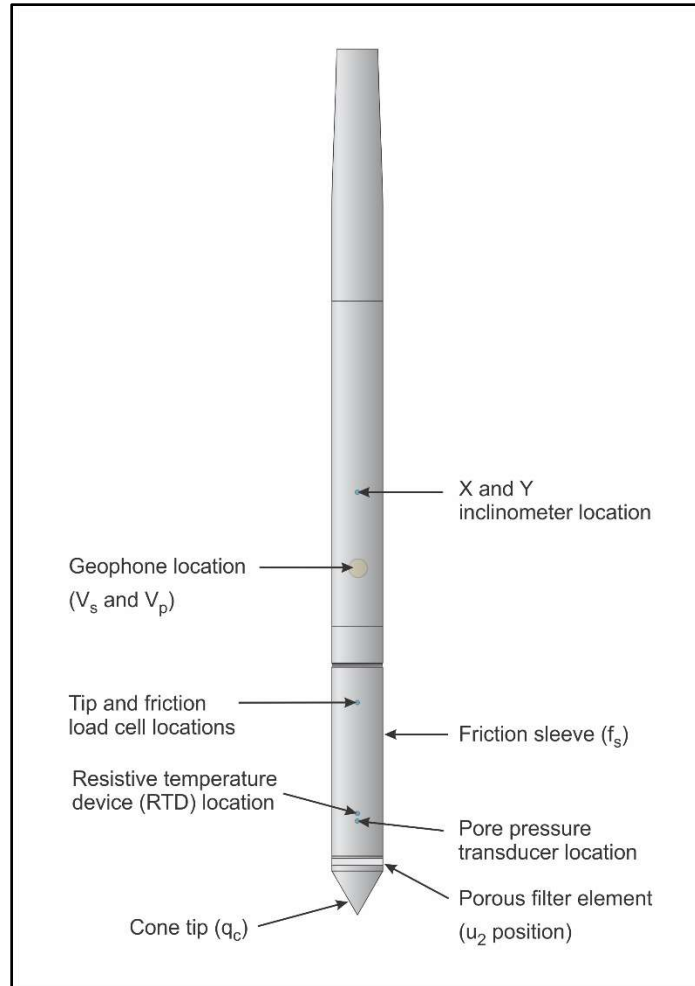


Figure CPTu. Piezocone Penetrometer (15 cm²)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPTu operating procedures which are in general accordance with the current [ASTM D5778](#) standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches (38.1 millimeters) are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with [ASTM](#) standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by [Robertson et al. \(1986\)](#) and [Robertson \(1990, 2009\)](#). It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in [Robertson et al. \(1986\)](#):

$$q_t = q_c + (1-a) \cdot u_2$$

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)

a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.



The friction ratio (R_f) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to [Robertson et al. \(1986\)](#), [Lunne et al. \(1997\)](#), [Robertson \(2009\)](#), [Mayne \(2013, 2014\)](#) and [Mayne and Peuchen \(2012\)](#).

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

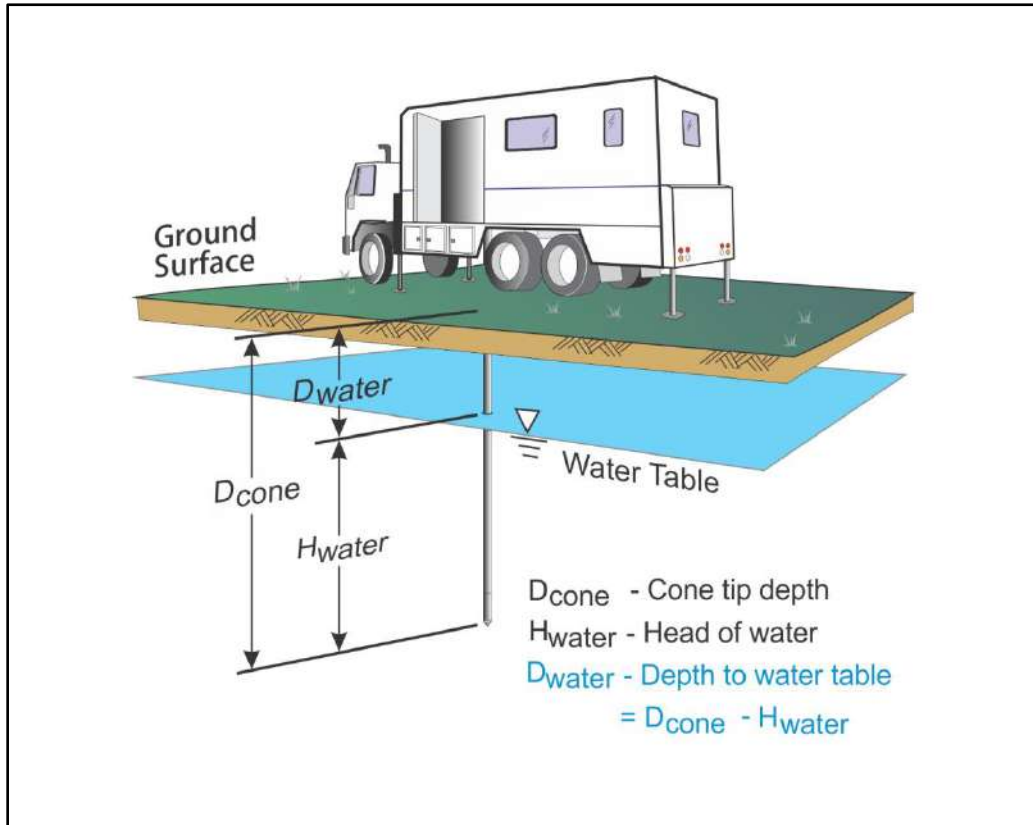


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

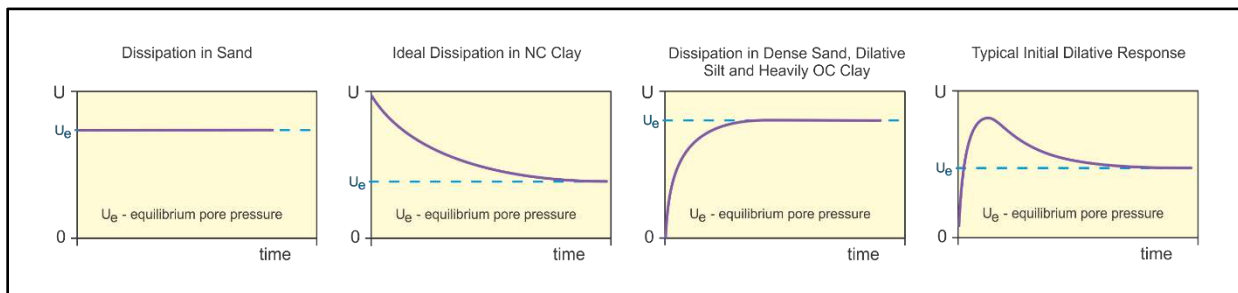


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in [Figure PPD-2](#).

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100} . In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100} . A theoretical analysis of pore pressure dissipations by [Teh and Houlsby \(1991\)](#) showed that a single curve relating degree of dissipation versus theoretical time factor (T^*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{l_r}}{t}$$

Where:

- T^* is the dimensionless time factor ([Table Time Factor](#))
- a is the radius of the cone
- l_r is the rigidity index
- t is the time at the degree of consolidation

Table Time Factor. T^* versus degree of dissipation ([Teh and Houlsby \(1991\)](#))

Degree of Dissipation (%)	20	30	40	50	60	70	80
$T^* (u_2)$	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50} , dissipation tests must be taken to a pressure less than u_{50} . The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100} . To estimate u_{50} , both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h ([Teh and Houlsby \(1991\)](#)), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (l_r) is assumed. For curves having an initial dilatatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50} . In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating l_r , the equilibrium pore pressure and the effect of an initial dilatatory response on calculating t_{50} , other methods should be applied to confirm the results for c_h .

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

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The following appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots
- Normalized Cone Penetration Test Plots
- SBT Zone Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots



Job No: 21-56-23177
Client: GeoPentech
Project: 1056 La Cienega Blvd
Start Date: 21-Oct-2021
End Date: 21-Oct-2021

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface ¹ (ft)	Final Depth (ft)	Northing ² (m)	Easting ² (m)	Elevation ³ (ft)	Refer to Notation Number
CPT-01	21-56-23177_CP01	21-Oct-2021	814:T1500F15U35	25.9	91.54	3769508	373021	124	
CPT-02	21-56-23177_CP02	21-Oct-2021	814:T1500F15U35	20.0	73.90	3769472	373023	125	4
CPT-03	21-56-23177_CP03	21-Oct-2021	814:T1500F15U35	18.5	66.03	3769450	373035	125	

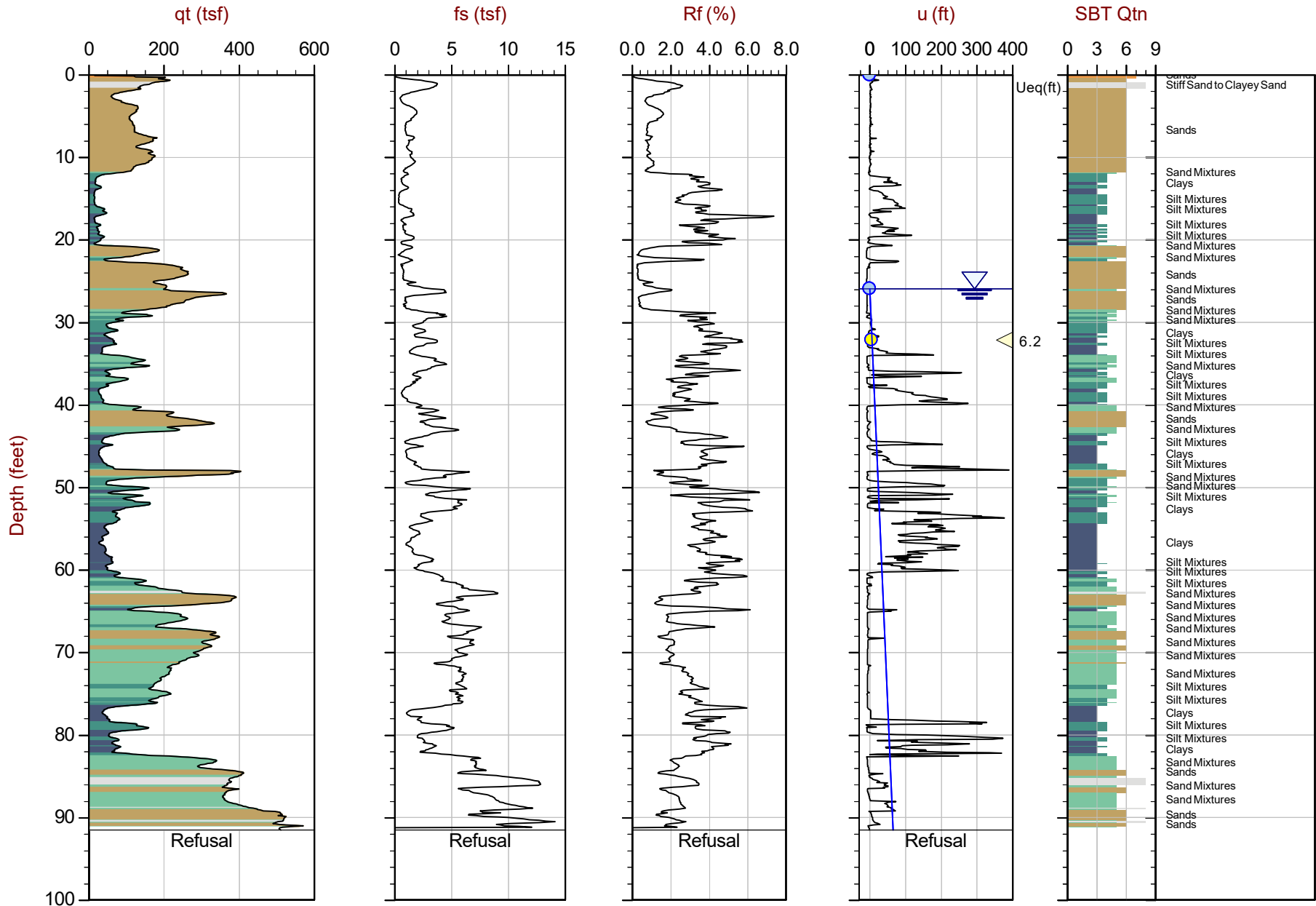
1. The assumed phreatic surface is based on the results of the pore pressure dissipation tests performed within or nearest each sounding. Hydrostatic conditions are assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 11S.
3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.
4. The assumed phreatic surface is based on the pore pressure dissipation test to not reach equilibrium within the sounding and the pore pressure dissipation tests at the nearby soundings.



GeoPentech

Job No: 21-56-23177
 Date: 2021-10-21 08:52
 Site: 1056 La Cienega Blvd

Sounding: CPT-01
 Cone: 814:T1500F15U35



Max Depth: 27.900 m / 91.53 ft
 Depth Inc: 0.025 m / 0.082 ft
 Avg Int: Every Point

File: 21-56-23177_CP01.COR
 Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
 Coords: UTM 11S N: 3769508m E: 373021m

● Equilibrium Pore Pressure (Ueq)
 ● Assumed Ueq
 ▲ Dissipation, Ueq achieved
 ▲ Dissipation, Ueq not achieved
 — Hydrostatic Line

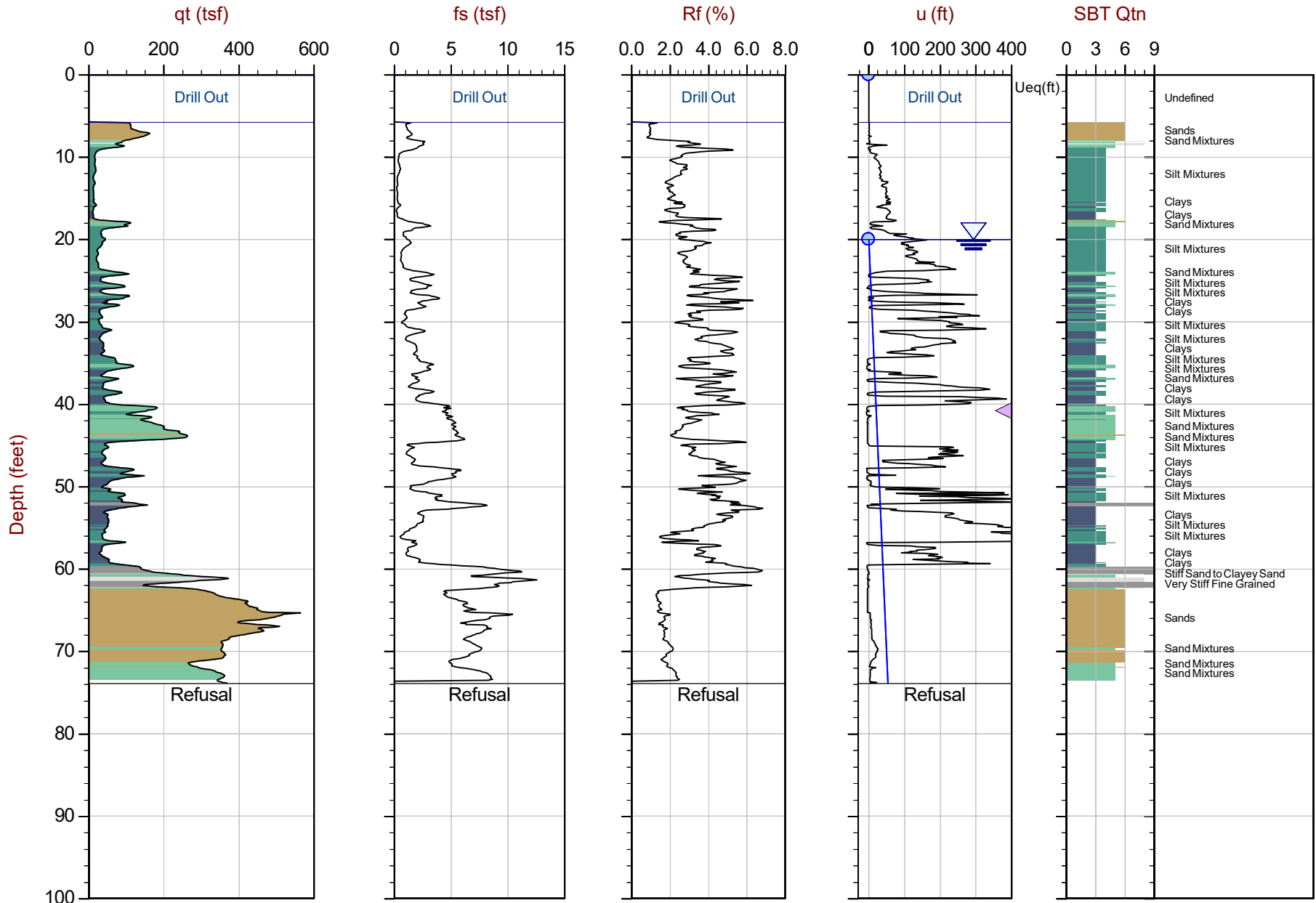
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



GeoPentech

Job No: 21-56-23177
 Date: 2021-10-21 10:53
 Site: 1056 La Cienega Blvd

Sounding: CPT-02
 Cone: 814:T1500F15U35



Max Depth: 22.525 m / 73.90 ft
 Depth Inc: 0.025 m / 0.082 ft
 Avg Int: Every Point

File: 21-56-23177_CP02.COR
 Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
 Coords: UTM 11S N: 3769472m E: 373023m

● Equilibrium Pore Pressure (Ueq)
 ● Assumed Ueq
 ◀ Dissipation, Ueq achieved
 ◀ Dissipation, Ueq not achieved
 — Hydrostatic Line

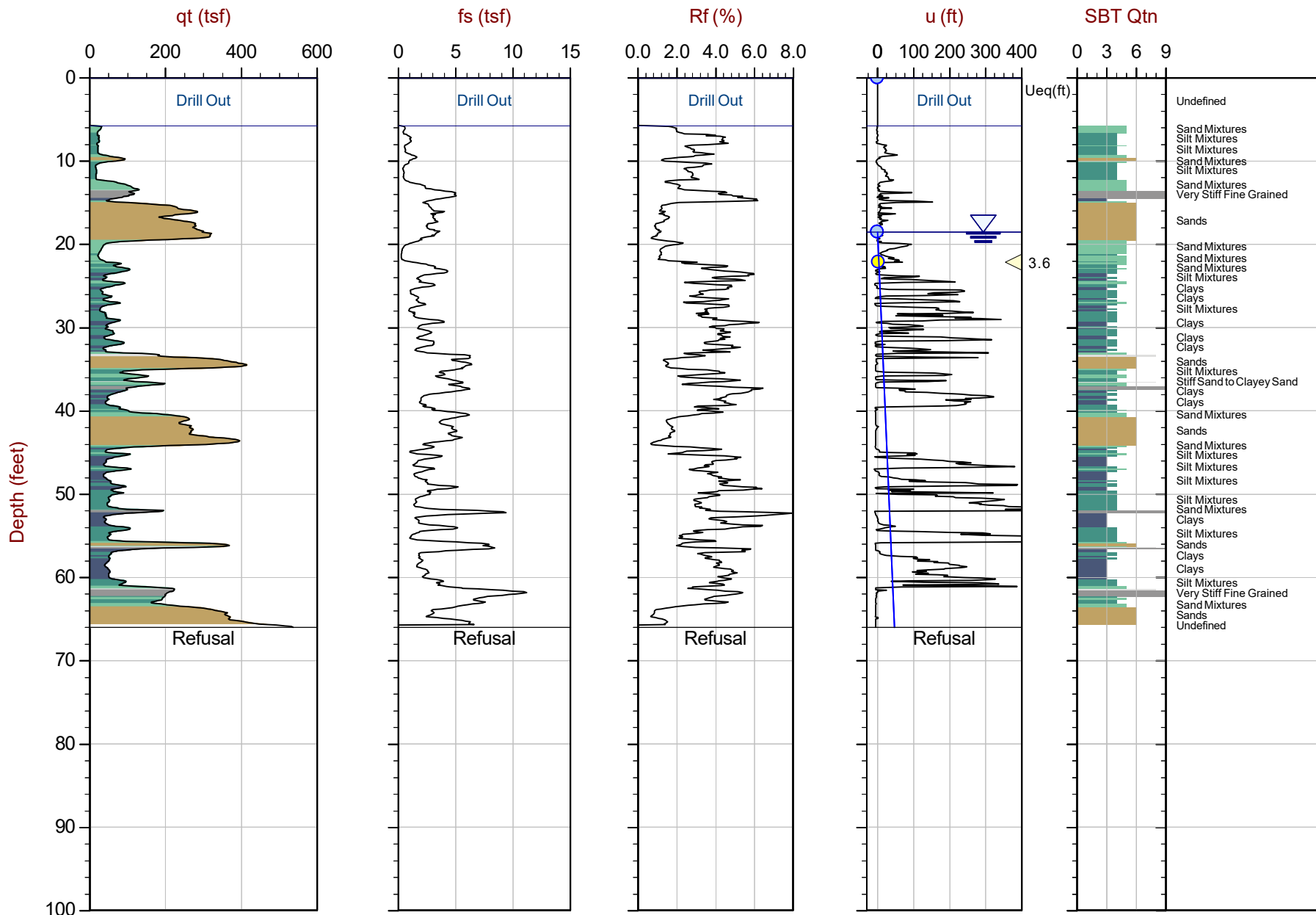
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



GeoPentech

Job No: 21-56-23177
Date: 2021-10-21 14:36
Site: 1056 La Cienega Blvd

Sounding: CPT-03
Cone: 814:T1500F15U35



Max Depth: 20.125 m / 66.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-23177_CP03.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769450m E: 373035m

● Equilibrium Pore Pressure (Ueq)
 ● Assumed Ueq
 ◀ Dissipation, Ueq achieved
 ◀ Dissipation, Ueq not achieved
 — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

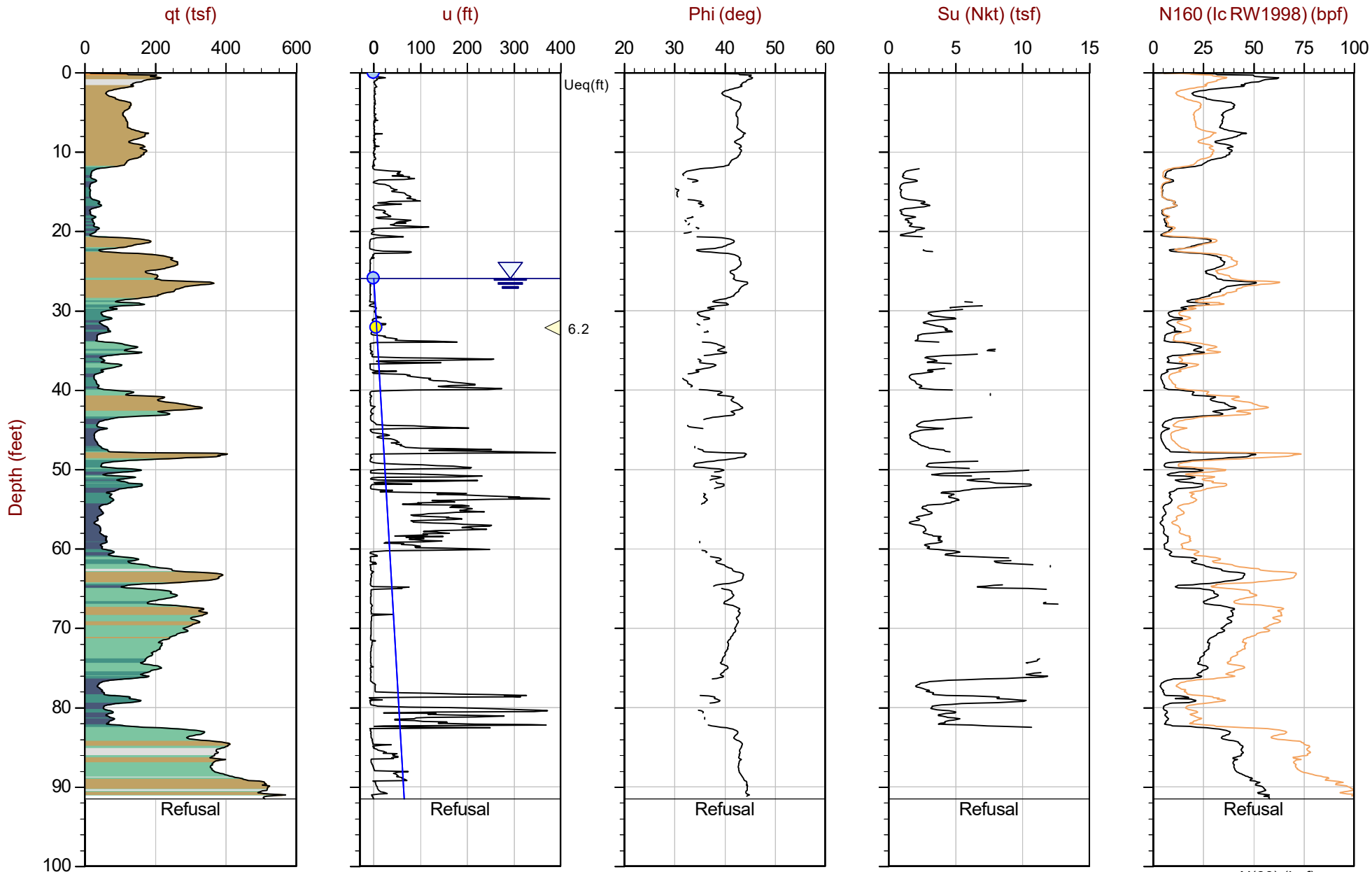
Advanced Cone Penetration Test Plots



GeoPentech

Job No: 21-56-23177
Date: 2021-10-21 08:52
Site: 1056 La Cienega Blvd

Sounding: CPT-01
Cone: 814:T1500F15U35



Max Depth: 27.900 m / 91.53 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-23177_CP01.COR
Unit Wt: SBTQtn(PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769508m E: 373021m

● Equilibrium Pore Pressure (Ueq)
 ● Assumed Ueq
 ◁ Dissipation, Ueq achieved
 ◁ Dissipation, Ueq not achieved
 — Hydrostatic Line
 — N(60) (bpf)

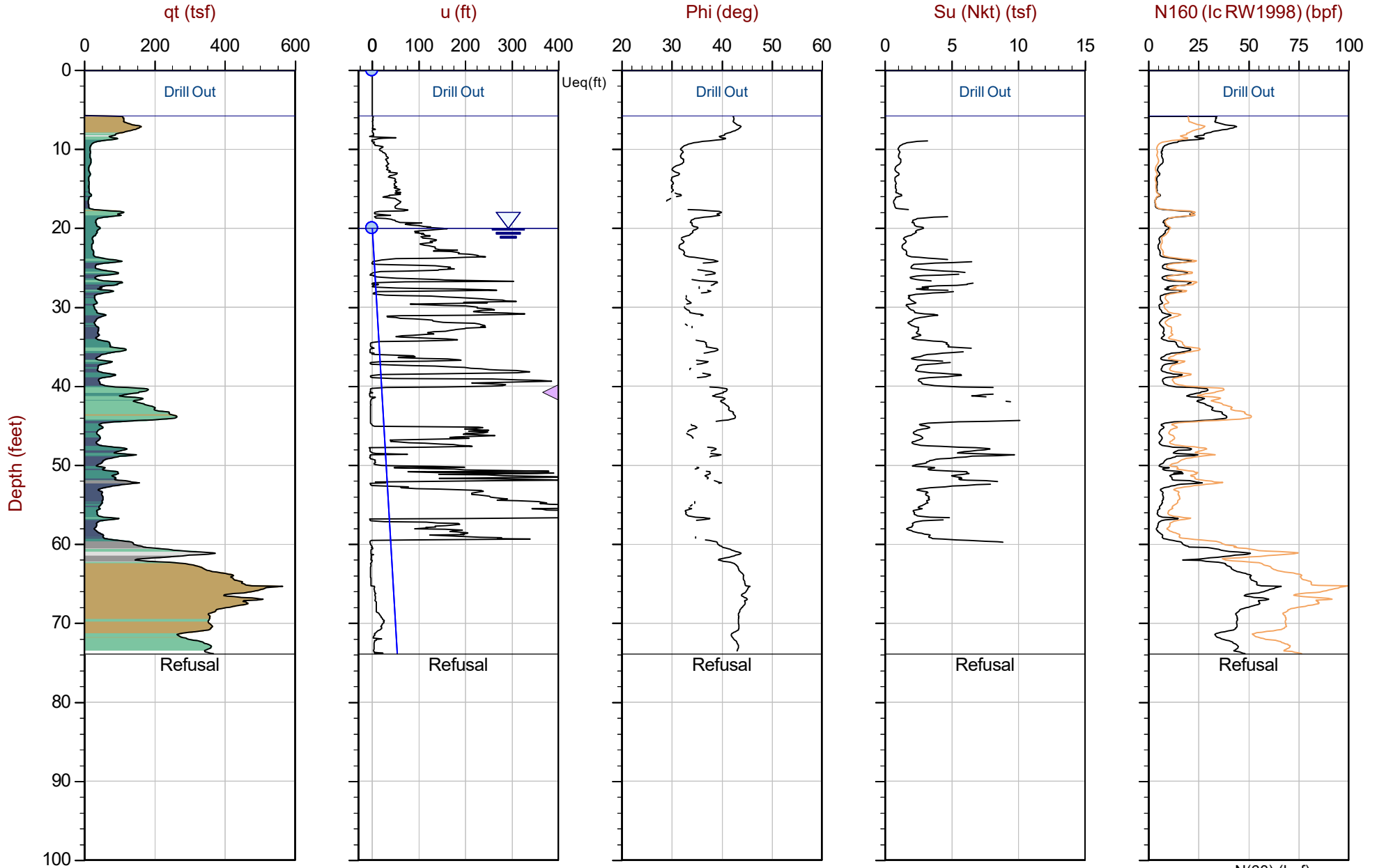
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



GeoPentech

Job No: 21-56-23177
Date: 2021-10-21 10:53
Site: 1056 La Cienega Blvd

Sounding: CPT-02
Cone: 814:T1500F15U35



Max Depth: 22.525 m / 73.90 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-23177_CP02.COR
Unit Wt: SBTQtn(PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769472m E: 373023m

- Equilibrium Pore Pressure (Ueq)
- Assumed Ueq
- ◀ Dissipation, Ueq achieved
- ◀ Dissipation, Ueq not achieved
- Hydrostatic Line

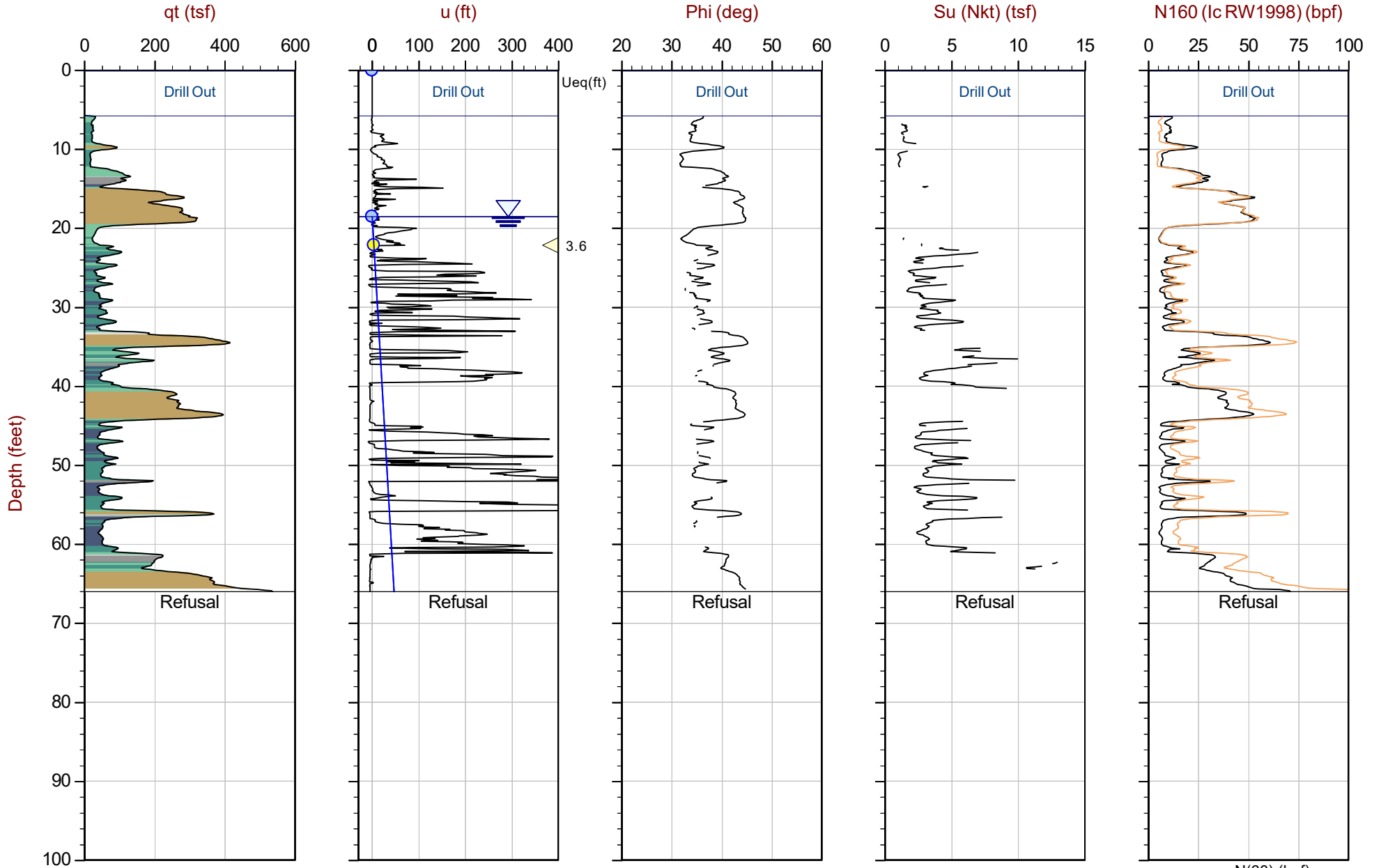
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



GeoPentech

Job No: 21-56-23177
Date: 2021-10-21 14:36
Site: 1056 La Cienega Blvd

Sounding: CPT-03
Cone: 814:T1500F15U35



Max Depth: 20.125 m / 66.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-23177_CP03.COR
Unit Wt: SBTQtn(PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769450m E: 373035m

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

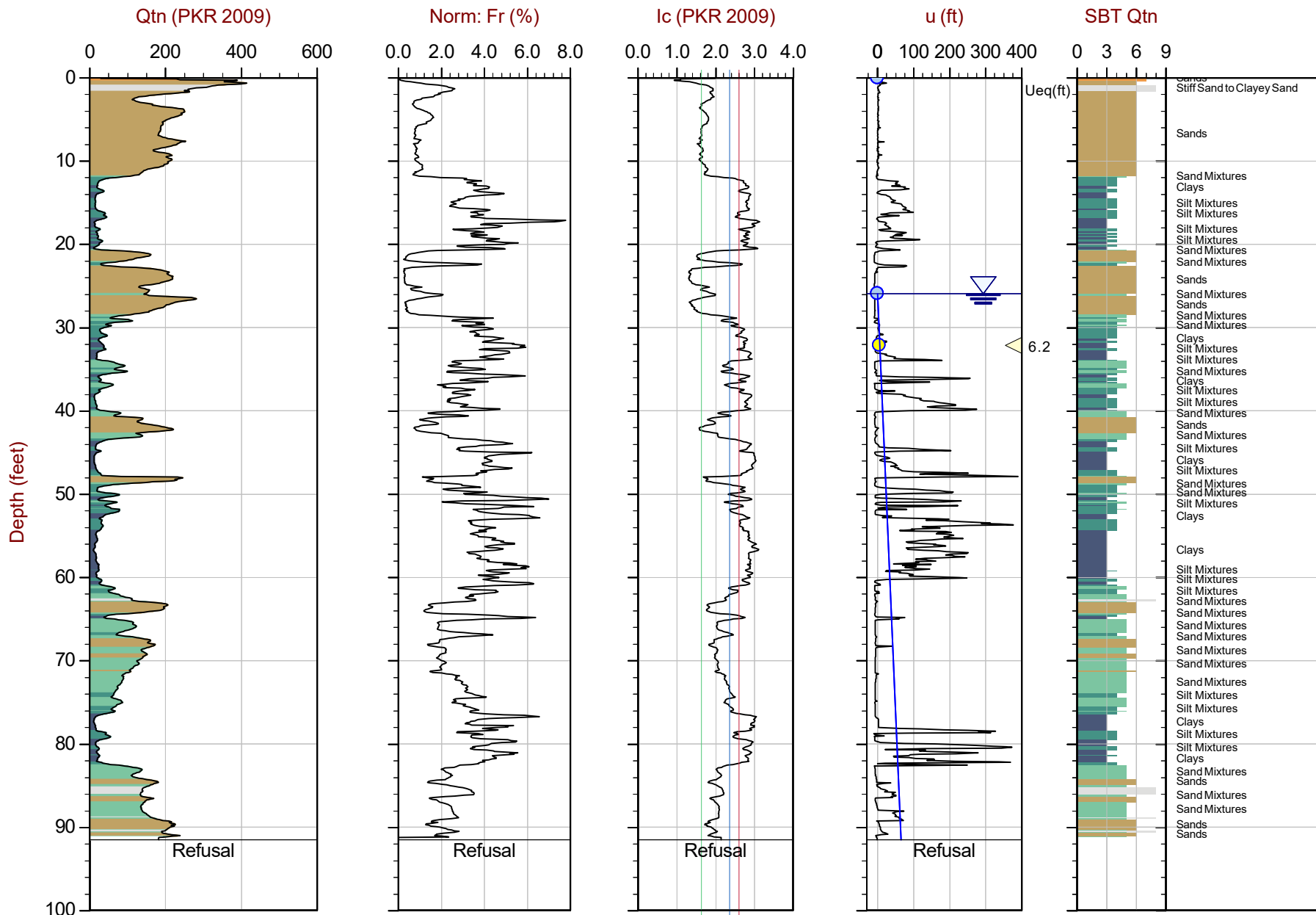
Normalized Cone Penetration Test Plots



GeoPentech

Job No: 21-56-23177
Date: 2021-10-21 08:52
Site: 1056 La Cienega Blvd

Sounding: CPT-01
Cone: 814:T1500F15U35



Max Depth: 27.900 m / 91.53 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-23177_CP01.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769508m E: 373021m

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

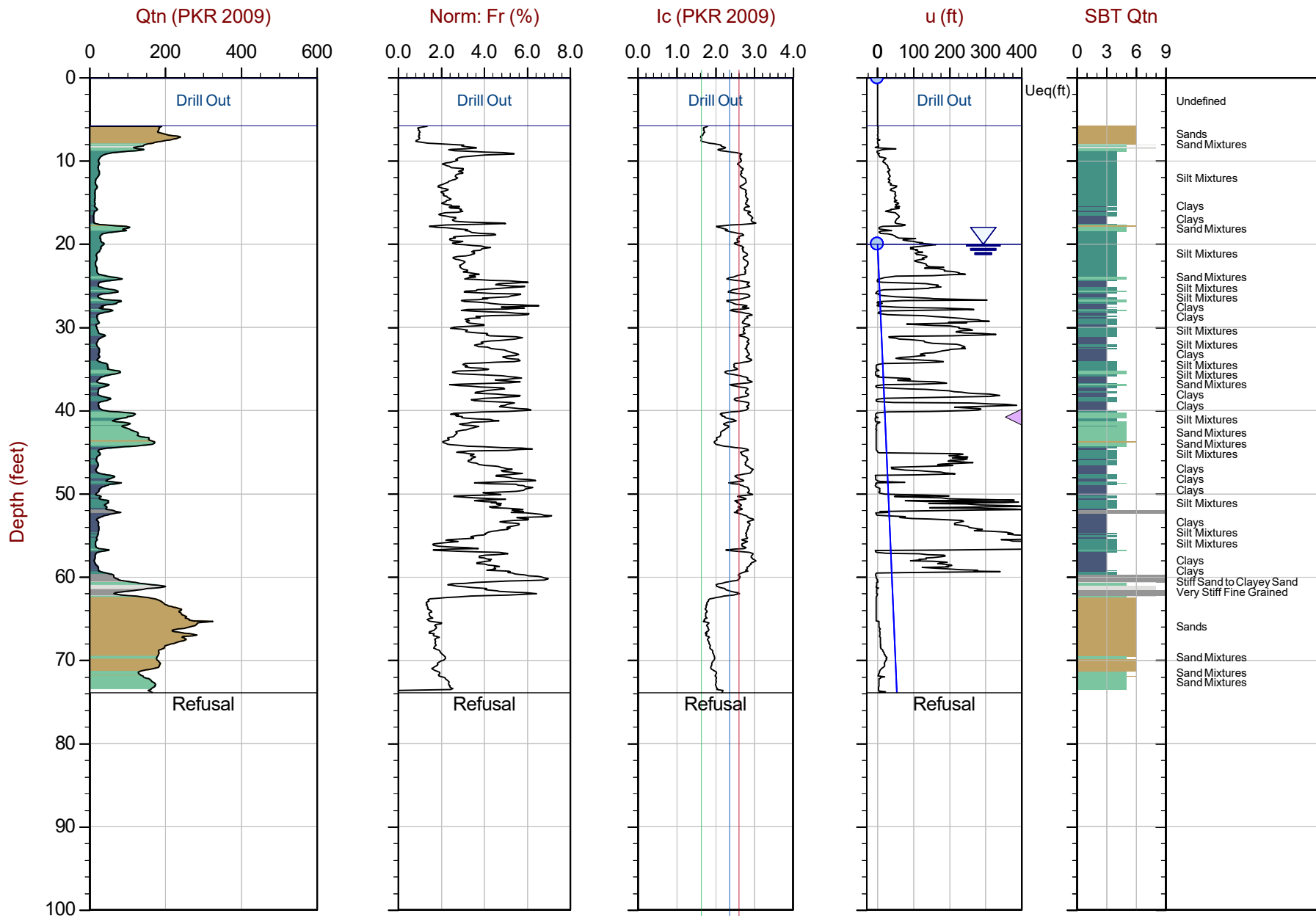
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



GeoPentech

Job No: 21-56-23177
Date: 2021-10-21 10:53
Site: 1056 La Cienega Blvd

Sounding: CPT-02
Cone: 814:T1500F15U35



Max Depth: 22.525 m / 73.90 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-56-23177_CP02.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769472m E: 373023m

● Equilibrium Pore Pressure (Ueq)
 ● Assumed Ueq
 ◀ Dissipation, Ueq achieved
 ◀ Dissipation, Ueq not achieved
 — Hydrostatic Line

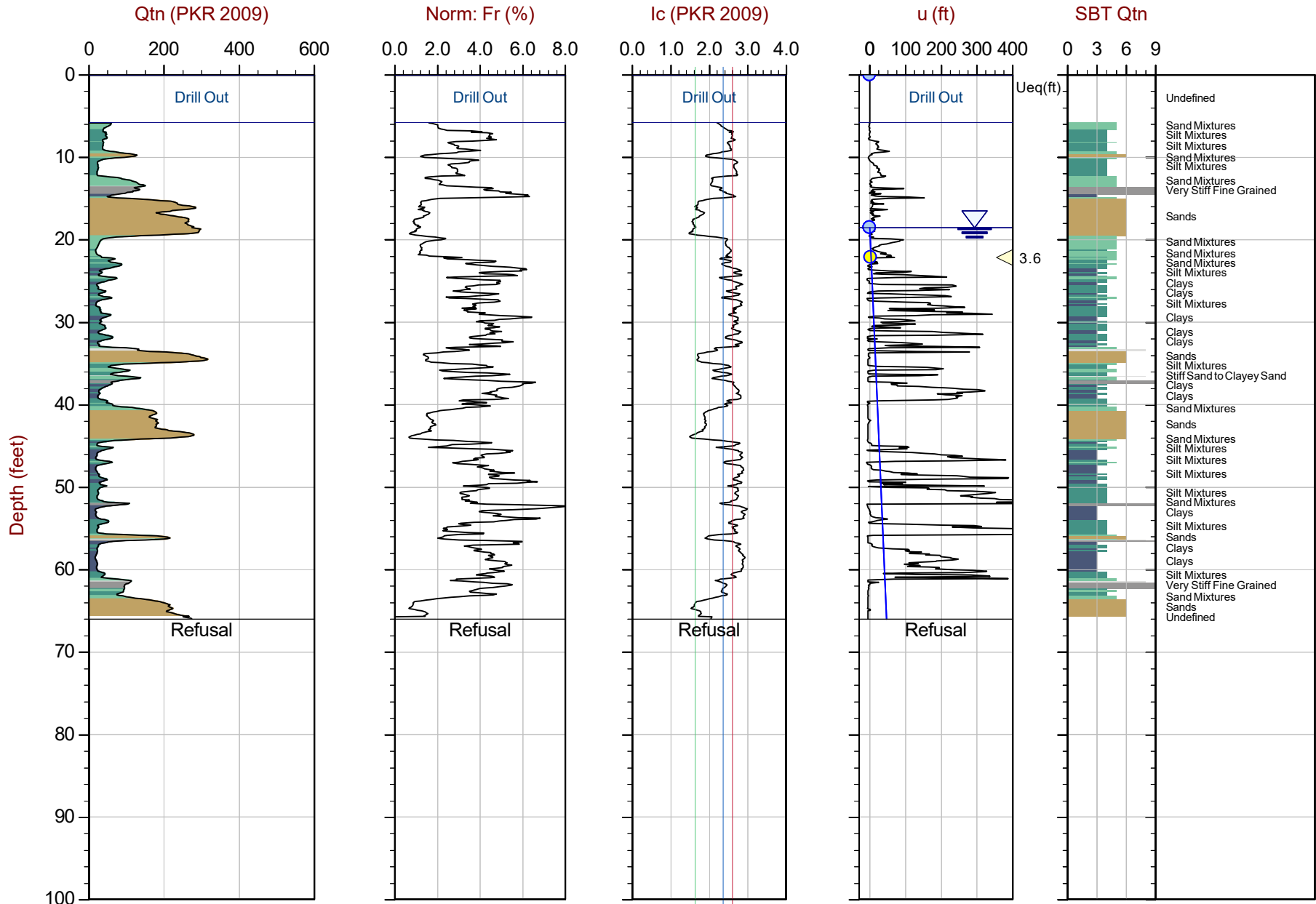
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



GeoPentech

Job No: 21-56-23177
Date: 2021-10-21 14:36
Site: 1056 La Cienega Blvd

Sounding: CPT-03
Cone: 814:T1500F15U35



Max Depth: 20.125 m / 66.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

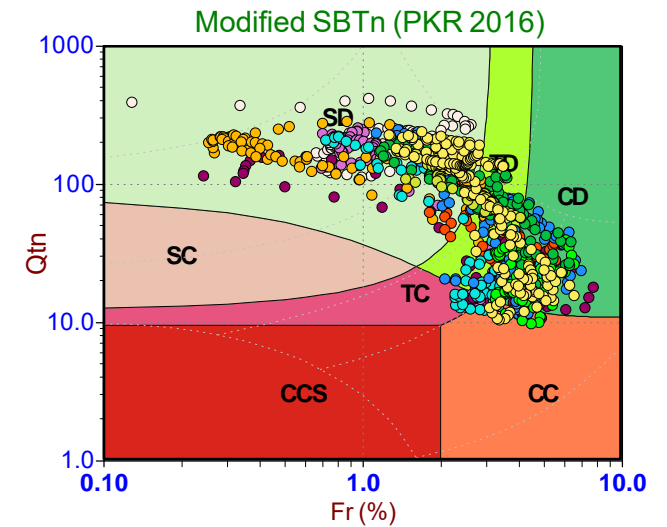
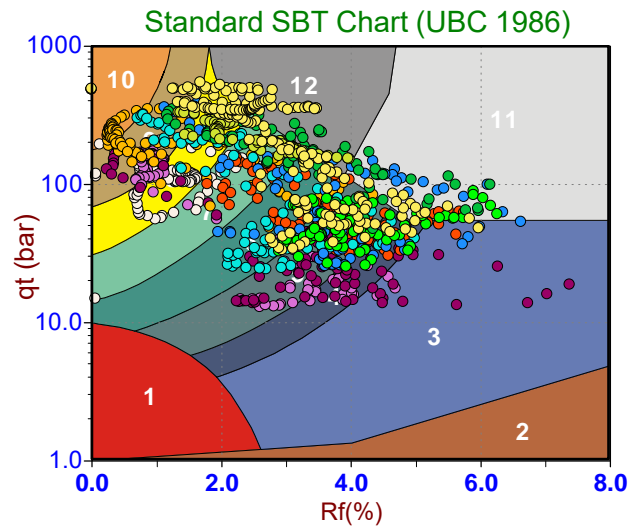
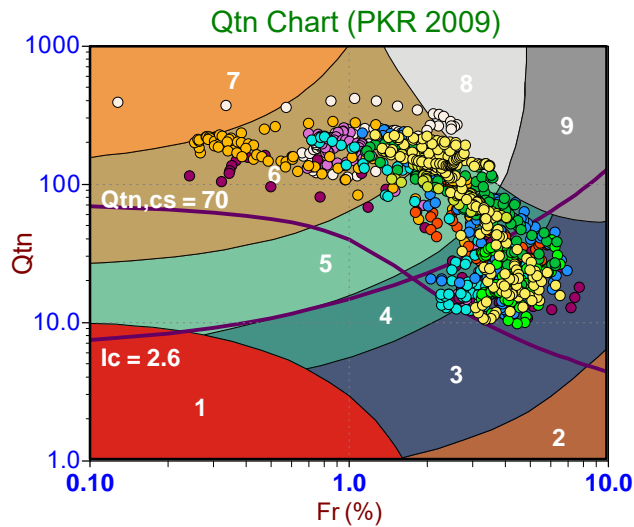
File: 21-56-23177_CP03.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 11S N: 3769450m E: 373035m

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Soil Behavior Type (SBT) Scatter Plots



Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend

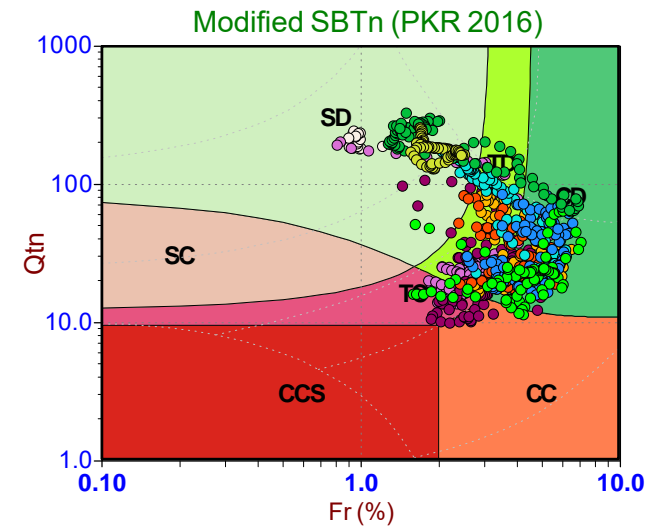
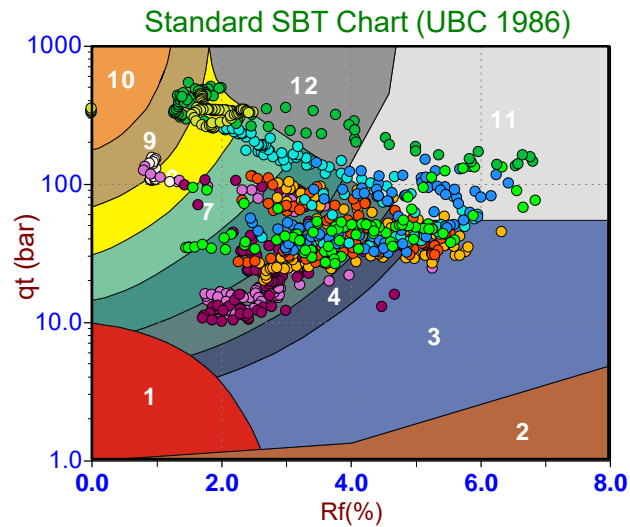
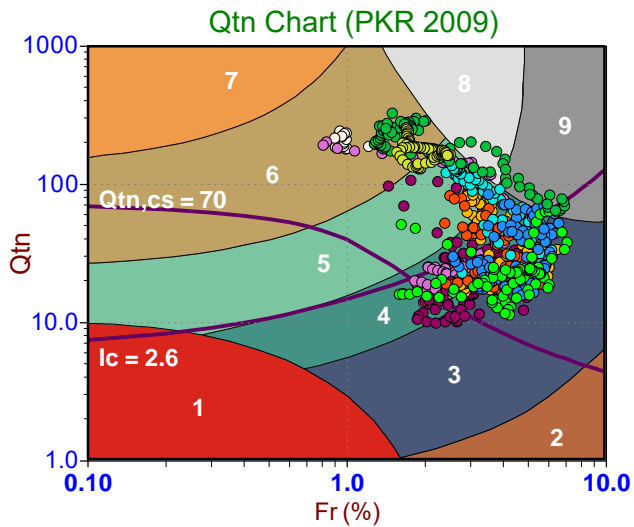
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)



Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend

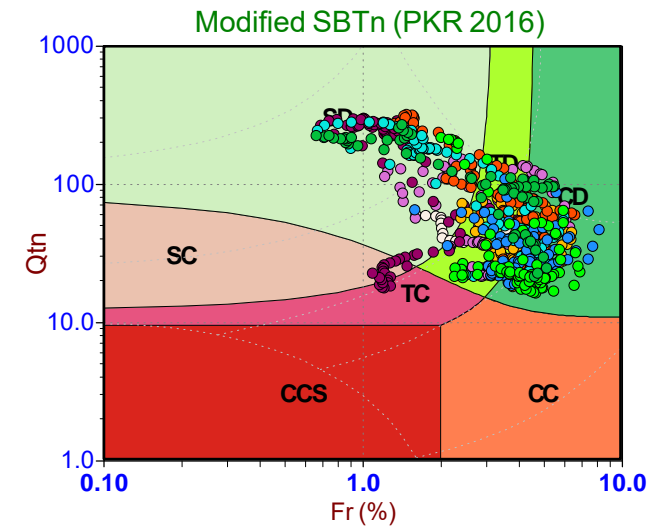
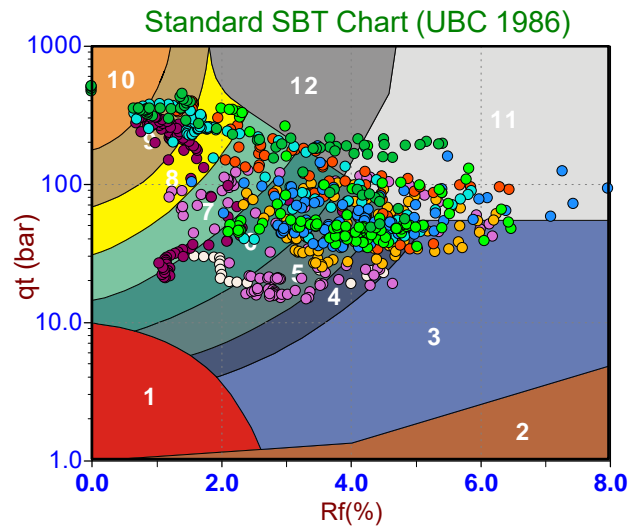
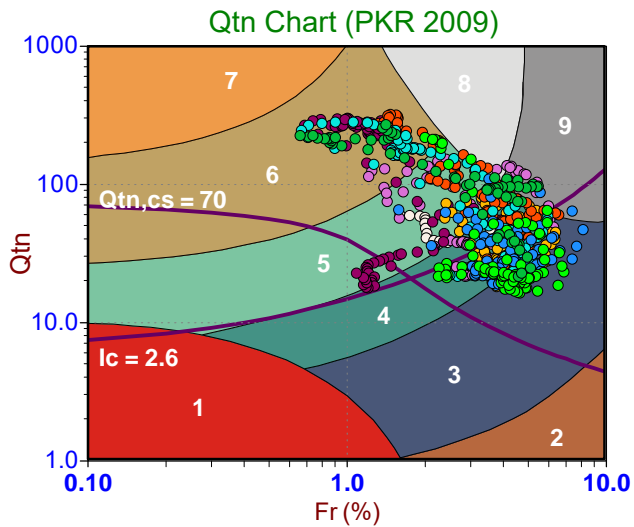
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)



Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



Job No: 21-56-23177
Client: GeoPentech
Project: 1056 La Cienega Blvd
Start Date: 21-Oct-2021
End Date: 21-Oct-2021

CPT_u PORE PRESSURE DISSIPATION SUMMARY

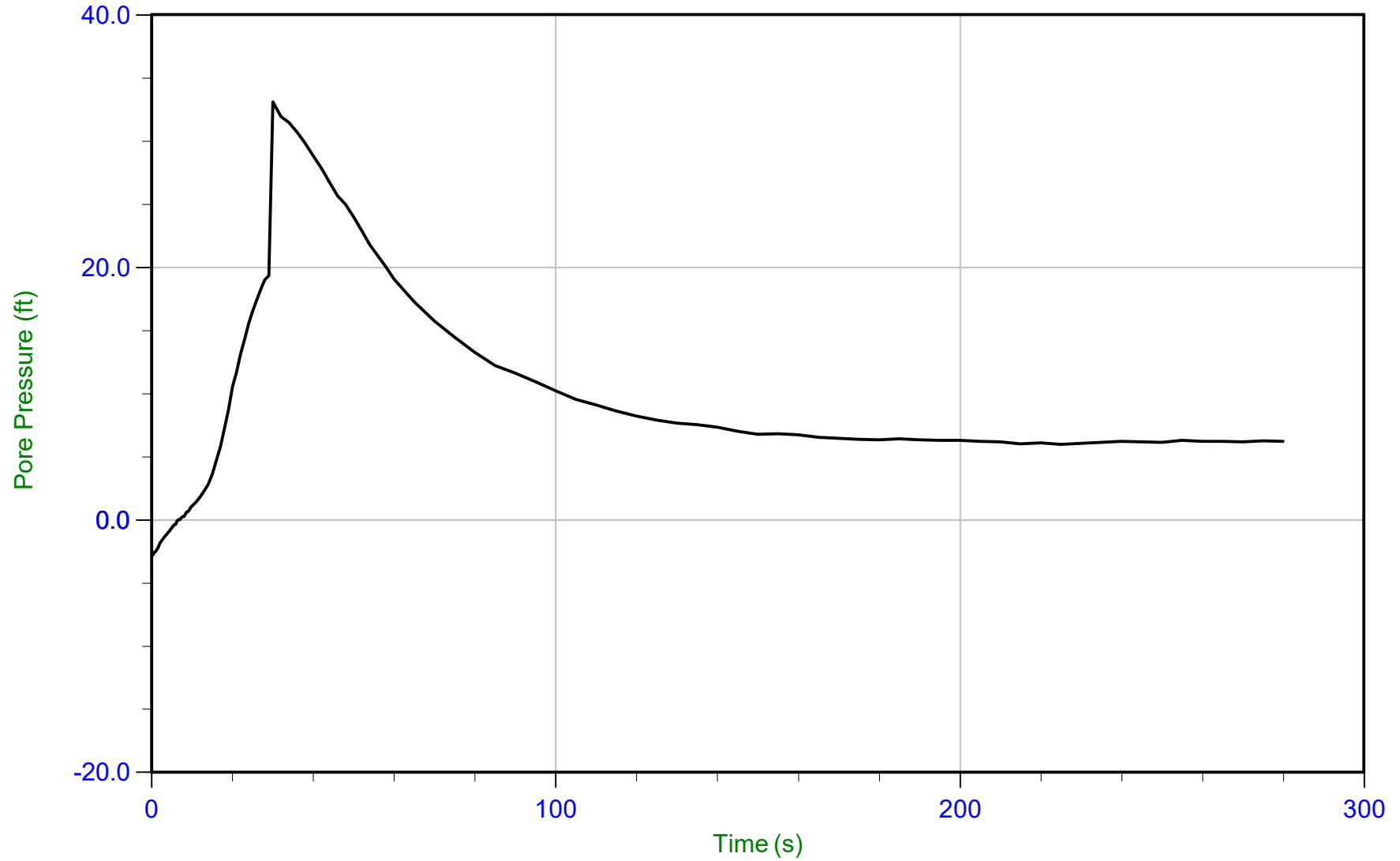
Sounding ID	File Name	Cone Area (cm ²)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U _{eq} (ft)	Calculated Phreatic Surface (ft)
CPT-01	21-56-23177_CP01	15	280	32.15	6.2	25.9
CPT-02	21-56-23177_CP02	15	360	40.76	Not Achieved	
CPT-03	21-56-23177_CP03	15	585	22.15	3.6	18.5



GeoPentech

Job No: 21-56-23177
Date: 10/21/2021 08:52
Site: 1056 La Cienega Blvd

Sounding: CPT-01
Cone: 814:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-56-23177_CP01.ppd2
Depth: 9.800 m / 32.152 ft
Duration: 280.0 s

u Min: -2.9 ft
u Max: 33.1 ft
u Final: 6.2 ft

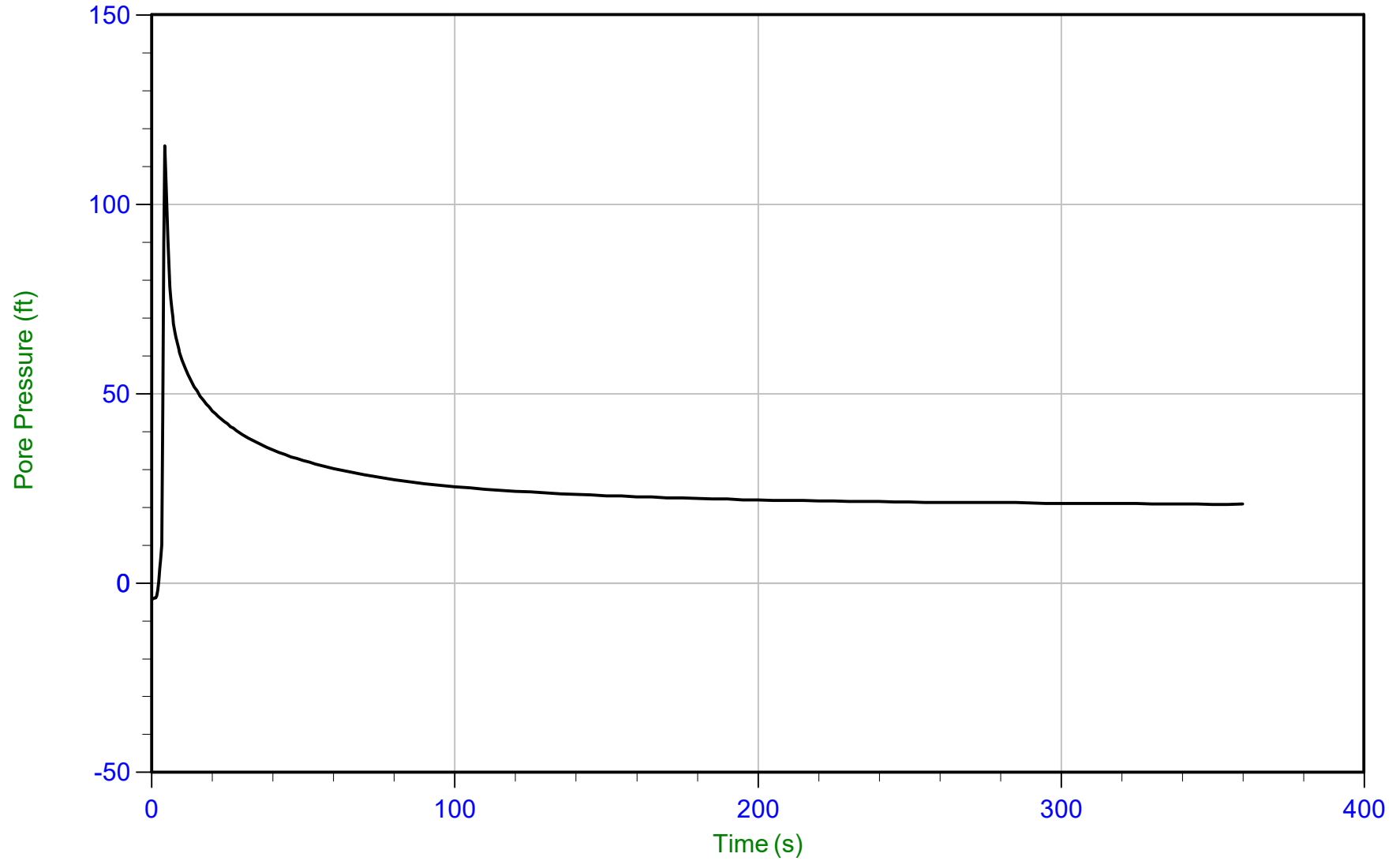
WT: 7.904 m / 25.931 ft
Ueq: 6.2 ft



GeoPentech

Job No: 21-56-23177
Date: 10/21/2021 10:53
Site: 1056 La Cienega Blvd

Sounding: CPT-02
Cone: 814:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-56-23177_CP02.ppd2
Depth: 12.425 m / 40.764 ft
Duration: 360.0 s

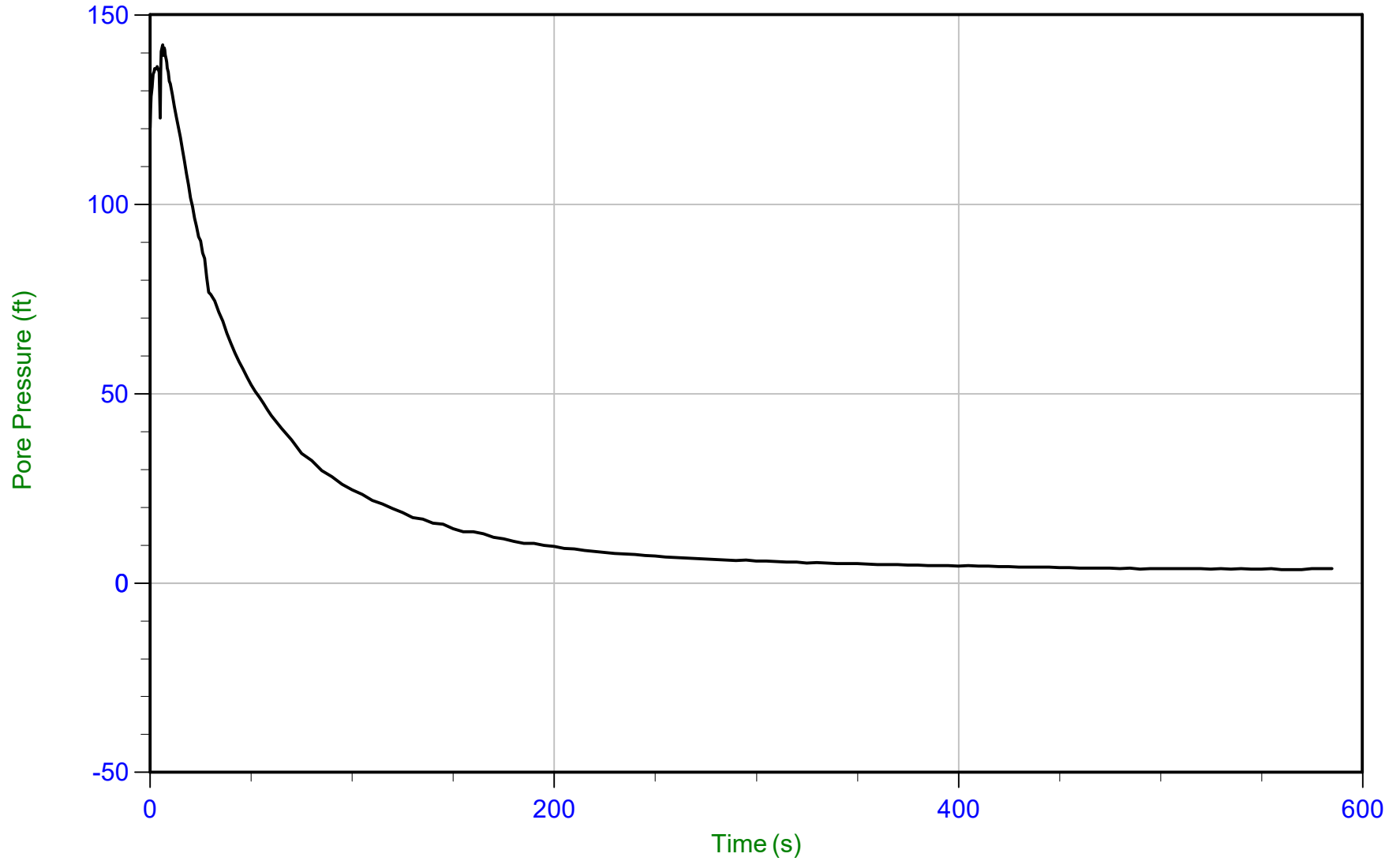
u Min: -4.2 ft
u Max: 115.4 ft
u Final: 20.8 ft



GeoPentech

Job No: 21-56-23177
Date: 10/21/2021 14:36
Site: 1056 La Cienega Blvd

Sounding: CPT-03
Cone: 814:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-56-23177_CP03.ppd2
Depth: 6.750 m / 22.145 ft
Duration: 585.0 s

u Min: 3.5 ft
u Max: 142.0 ft
u Final: 3.8 ft

WT: 5.650 m / 18.536 ft
Ueq: 3.6 ft

APPENDIX D

LABORATORY TESTING

D.1 LABORATORY TESTING

The laboratory testing program performed by GeoPentech for the proposed project site included the following tests: moisture content, dry density, sieve analysis, wash analysis, direct shear, consolidation, and corrosion. The geotechnical testing was conducted at the laboratory facilities of AP Engineers in Pomona, California. The tests were performed in general accordance with applicable procedures of ASTM and the State of California Department of Transportation, Standard Test Methods (DOT CA). The results of the laboratory testing are included in this Appendix and are summarized in Table D-1 and on the boring logs in Appendix B. GeoPentech has reviewed the results of the laboratory testing and finds them acceptable. Brief descriptions of each test are presented in the following sections.

D.1.1 Moisture Content and Dry Density

For selected Modified California samples, the dry unit weight (in units of pounds-per-cubic-foot) and field moisture content (%) were measured in general accordance with ASTM D2937 and ASTM D2216, respectively, or with ASTM D7263.

D.1.2 Sieve Analysis and Wash Analysis

For selected samples, the particle-size distribution was determined by sieve analysis in general accordance with ASTM D6913. Sieve sizes ranged from $\frac{3}{4}$ in to 75 μm (No. 200).

For other selected samples, the percentage of fines (material passing the No. 200 sieve) was measured by wash analysis in accordance with ASTM D1140.

D.1.3 Atterberg Limits

The Atterberg limits test is a classification test that is performed on cohesive soils (i.e., silty and clayey soils) to measure the soil plastic limit (PL) and liquid limit (LL), from which the plasticity index (PI) is calculated. The measured values can be plotted on a plasticity chart, which is used as an aid in classifying the soil material and behavior. These tests were performed in accordance with ASTM D4318.

D.1.4 Corrosion Tests

Soil samples were tested for electrical resistivity, pH, sulfate content, and chloride content. These tests were performed in general accordance with DOT CA test methods 643 (electrical resistivity and pH), 417 (sulfate content), and 422 (chloride content). The test results were used to evaluate the corrosivity potential of the soil on underground improvements associated with the proposed structure.

D.1.5 Direct Shear

Direct shear tests were performed on selected Modified California samples in accordance with ASTM D3080 to measure peak and ultimate strength parameters. Shear stress and sample deformation were monitored throughout the tests.

D.1.6 Consolidation

Tests for one-dimensional consolidation properties of soils using incremental loading were performed on relatively undisturbed soil samples according to ASTM D2435. The test determines the magnitude and rate of consolidation of soil when it is restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. The test results provide clayey soil settlement parameters under different loading conditions.

Table D-1
Summary of Laboratory Testing

Boring Number	Location		Classification	Initial Condition		Atterberg		Gradation			Corrosion				Peak Strength (DS)		Other Tests
	Sample No.	Depth (ft)		Moisture (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Min. Resistivity (Ohm-cm)	Sulfate Cont. (ppm)	Chloride Cont. (ppm)	pH	Friction Angle (Degrees)	Cohesion (psf)	
GP-1	Bulk-1	0-5	SW-SM								3863	62	56	8.3			
GP-1	1	2.5	SW-SM														
GP-1	2a	5	SW-SM	5.0	104.0					7.6							
GP-1	2b	5	SW-SM					14	79	7							
GP-1	3a	7.5	SW-SM														
GP-1	3b	7.5	SW-SM														
GP-1	4a	10	SP-SM	13.8				5	89	6							
GP-1	4b	10	SP-SM														
GP-1	5	12	SP-SM														
GP-1	6a	16	SP					4	93	3				38	700		
GP-1	6b	16	SP														
GP-1	7	19	SM	18.5		NP	NP			12.2							
GP-1	8	23	SC														
GP-1	9a	29	SC	17.2	108.1	47	25			34.9							CONSOL
GP-1	9b	29	CL											28	800		
GP-1	10	34	CL	26.4		30	12										
GP-1	11a	39	SC	26.1													
GP-1	11b	39	SP-SC														
GP-1	12	44	CL	31.8		43	31			84.1							
GP-1	13a	49	CL	27.9	98.7	28	13			88.8							CONSOL
GP-1	13b	49	CL														
GP-1	14	54	CL			35	19			66.8							
GP-1	15a	59	CL	25.6	104.6	44	28			70.3							
GP-1	15b	59	CL														CONSOL
GP-1	16	64	SM	26.6						44.7							
GP-1	17	69	SM														
GP-1	18	74	SM														
GP-1	19	79	CH	38.1		51	25			96.2							
GP-1	20a	84	ML	27.4	101.2					65.9							
GP-1	20b	84	ML														
GP-1	21a	94	SM							49.2							
GP-1	21b	94	SM														
GP-1	22	99	CH			64	45			95.1							

Table D-1
Summary of Laboratory Testing

Boring Number	Location		Classification	Initial Condition		Atterberg		Gradation			Corrosion				Peak Strength (DS)		Other Tests
	Sample No.	Depth (ft)		USCS Symbol	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Min. Resistivity (Ohm-cm)	Sulfate Cont. (ppm)	Chloride Cont. (ppm)	pH	Friction Angle (Degrees)	
GP-2	Bulk-1	0-5	CL														
GP-2	1a	2.5	CH	11.5	117.7	50	37	1	45	54							
GP-2	1b	2.5	CH														
GP-2	2	5	CL			41	27										
GP-2	3a	7.5	CL	17.9	108.6					57.9				36	850		
GP-2	3b	7.5	CL					0	44	56							
GP-2	4	10	CL														
GP-2	5a	12	ML	28.3	94.1					67.7							
GP-2	5b	12	ML														
GP-2	6	14	ML														
GP-2	7a	17	SC	11.0	109.5	53	37	14	61	25							
GP-2	7b	17	SC														
GP-2	8	19	CL														
GP-2	9a	24	CL	31.3	92.1	41	16			97.2				29	500	CONSOL	
GP-2	9b	24	CL														
GP-2	10	29	CL			34	12										
GP-2	11a	34	SM							43.4							
GP-2	11b	34	SM	21.8	106.0	NP	NP			27							
GP-2	12	39	ML														
GP-2	13a	44	CL	31.1	92.0	41	23			95.6							CONSOL
GP-2	13b	44	CL														
GP-2	14a	49	ML							55.1							
GP-2	14b	49	ML														
GP-3	Bulk-1	0-5	SM														
GP-3	1a	5	SW-SM														
GP-3	1b	5	SW-SM														
GP-3	2a	10	CH			55	38			94.7							
GP-3	2b	10	CH														
GP-3	3a	14	SM							83.3							
GP-3	3b	14	SM														
GP-3	4a	19	CH			55	36										
GP-3	4b	19	CL	22.0	94.1									29	300		
GP-3	5a	24	CL							96.3							

Table D-1
Summary of Laboratory Testing

Boring Number	Location		Classification	Initial Condition		Atterberg		Gradation			Corrosion				Peak Strength (DS)		Other Tests
	Sample No.	Depth (ft)		USCS Symbol	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Min. Resistivity (Ohm-cm)	Sulfate Cont. (ppm)	Chloride Cont. (ppm)	pH	Friction Angle (Degrees)	
GP-3	5b	24	CH	30.9	88.3										18	900	
GP-3	6	29	CH			58	43	0	9	91							
GP-3	7a	32	ML							69.9							
GP-3	7b	32	ML														
GP-3	8a	34	SP-SM							19.7							
GP-3	8b	34	SP-SM														
GP-3	9a	39	SP-SM							19.8							
GP-3	9b	39	SP-SM	21.2	105.5												
GP-3	10a	44	ML														
GP-3	10b	44	ML														
GP-3	11	45.5	CL	20.4	109.1	37	19			71.1							CONSOL
GP-3	12a	49	CL														
GP-3	12b	49	CL														
GP-3	13a	54	CL	21.6	105.1	30	18			55.5							CONSOL
GP-3	13b	54	CL														
GP-3	14	55.5	CL	24.2	102.5	44	25			63.6							CONSOL
GP-3	15a	59	SP-SM							32.4							
GP-3	15b	59	SP-SM														
GP-3	16a	64	SP-SM														
GP-3	16b	64	SP-SM														
GP-3	17a	69	SP-SM														
GP-3	17b	69	SP-SM														
GP-3	18a	74	SP-SM														
GP-3	18b	74	SP-SM														
GP-3	19a	79	SP-SM			NP	NP			87.1							
GP-3	19b	79	SP-SM														
GP-3	20a	84	SP-SM														
GP-3	20b	84	SP-SM														
GP-3	21a	89	SP-SM														
GP-3	21b	89	SP-SM														
GP-3	22a	99	CH	32.3	90.6	73	53			97.7							CONSOL
GP-3	22b	99	CH														
GP-3	23-top	104.5	CH	28.6	94.2	51	32			87							CONSOL

Table D-1
Summary of Laboratory Testing

Boring Number	Location		Classification	Initial Condition		Atterberg		Gradation			Corrosion				Peak Strength (DS)		Other Tests
	Sample No.	Depth (ft)		USCS Symbol	Moisture (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Min. Resistivity (Ohm-cm)	Sulfate Cont. (ppm)	Chloride Cont. (ppm)	pH	Friction Angle (Degrees)	
GP-3	23-bottom	104.5	CH														
GP-3	24a	109	CL			42	27			53							
GP-3	24b	109	CL														
GP-3	25a	114	CL							43.8							
GP-3	25b	114	CL														
GP-3	26a	119	CL														
GP-3	26b	119	CL														
GP-3	27a	129	CL			41	27										
GP-3	27b	129	CL														
GP-3	28a	139	CH	28.2	96.9	51	32										CONSOL
GP-3	28b	139	CH														
GP-3	29	149.5	SP-SM							6.7							
GP-3	30	184	SP-SM														
GP-3	31	199	SP-SM														
GP-4	1	5	SC														
GP-4	2	10	SM														
GP-4	3	15	SW														
GP-4	4	20	SM							24.1							
GP-4	5	25	CL														
GP-4	6	30	CL	23.8	101.8	33	10			69.9							
GP-4	7	35	CL														
GP-4	8	40	SC							47.1							
GP-4	9	45	CL	24.2	102.9	38	21			90.4							CONSOL
GP-4	10	50	CL											23	1300		
GP-4	11	55	SC							27.8							
GP-4	12	60	SP														
GP-4	13	65	SM							18.1							
GP-4	14	70	SP											35	750		
GP-4	15	75	SC														
GP-4	16	80	CH	32.4	93.6	51	30			85.3							CONSOL
GP-4	17	85	CL														
GP-4	18	90	SM														
GP-4	19	95	ML														

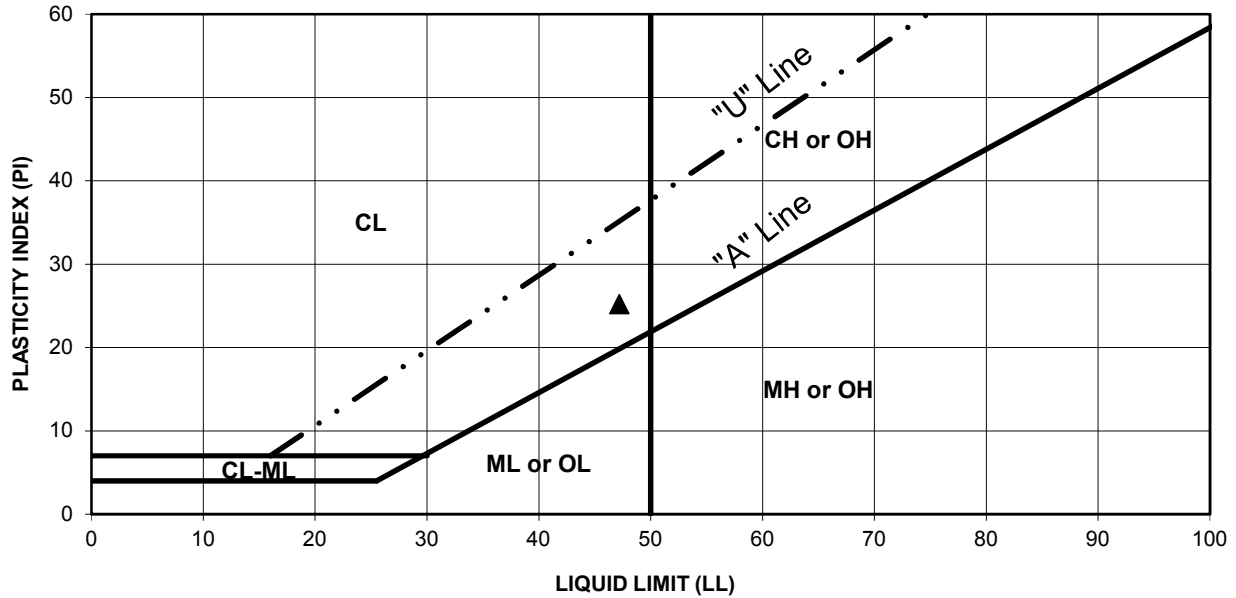
Table D-1
Summary of Laboratory Testing

Boring Number	Location		Classification	Initial Condition		Atterberg		Gradation			Corrosion				Peak Strength (DS)		Other Tests
	Sample No.	Depth (ft)		Moisture (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Min. Resistivity (Ohm-cm)	Sulfate Cont. (ppm)	Chloride Cont. (ppm)	pH	Friction Angle (Degrees)	Cohesion (psf)	
GP-4	20	100	ML	29.3	94.6	NP	NP			93							
GP-4	21	105	SC														
GP-4	22	110	ML	28.1	97.6					62.9				31	1050		
GP-4	23	115	CL														
GP-4	24	120	SC	16.3	116.9	36	22			31.1							
GP-4	25	125	CL														
GP-4	26	130	SC			23	9			32.3							
GP-4	27	135	SC														
GP-4	28	140	CL	37.0	92.2	41	24			94.6							
GP-4	29	145	CL														
GP-4	30	150	CL	28.5	94.8	45	27			58.3							
GP-4	31	155	CH	30.4	95.7	60	45			50.3						CONSOL	
GP-4	32	160	SP-SM							7.6							
GP-4	33	165	NR														
GP-4	34	170	SP-SM														
GP-4	35	175	SP-SM														



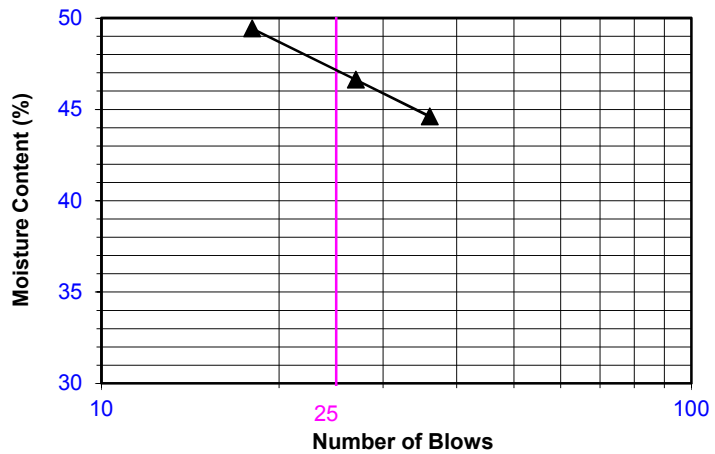
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>NG</u>	Date: <u>09/02/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>09/03/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>09/13/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test



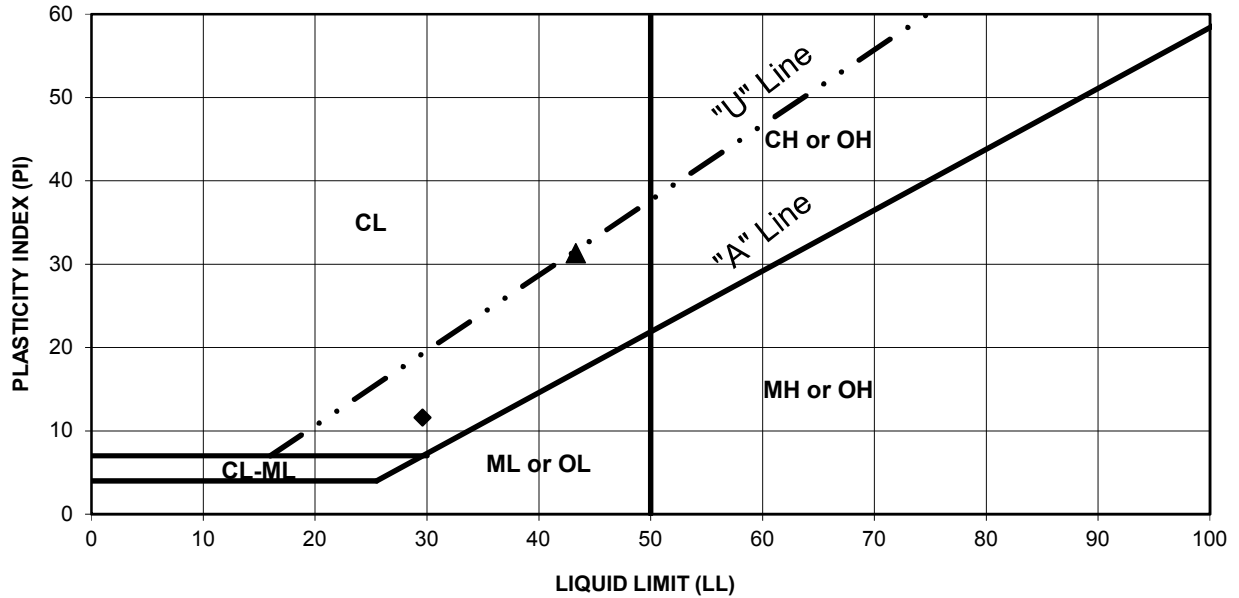
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	GP-1	7	19	NP	NP	NP	
▲	GP-1	9a	29	47	22	25	CL

* NP denotes "non-plastic"



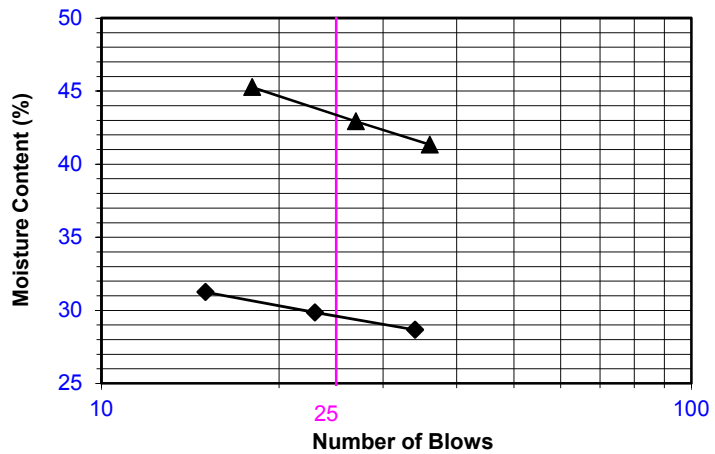
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>NG</u>	Date: <u>09/02/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>09/03/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>09/13/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test

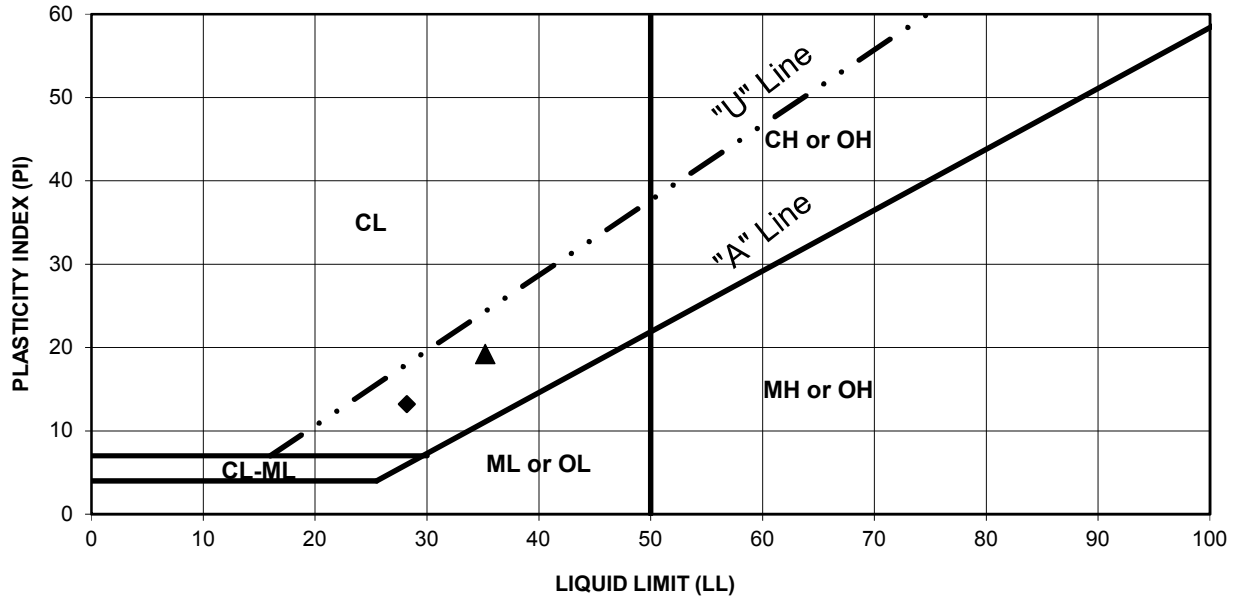


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-1	10	34	30	18	12	CL
▲	GP-1	12	44	43	12	31	CL



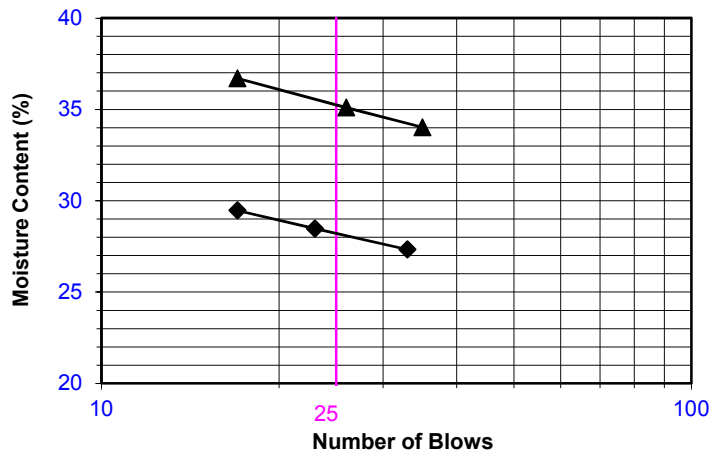
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>NG</u>	Date: <u>09/02/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>09/03/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>09/13/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test

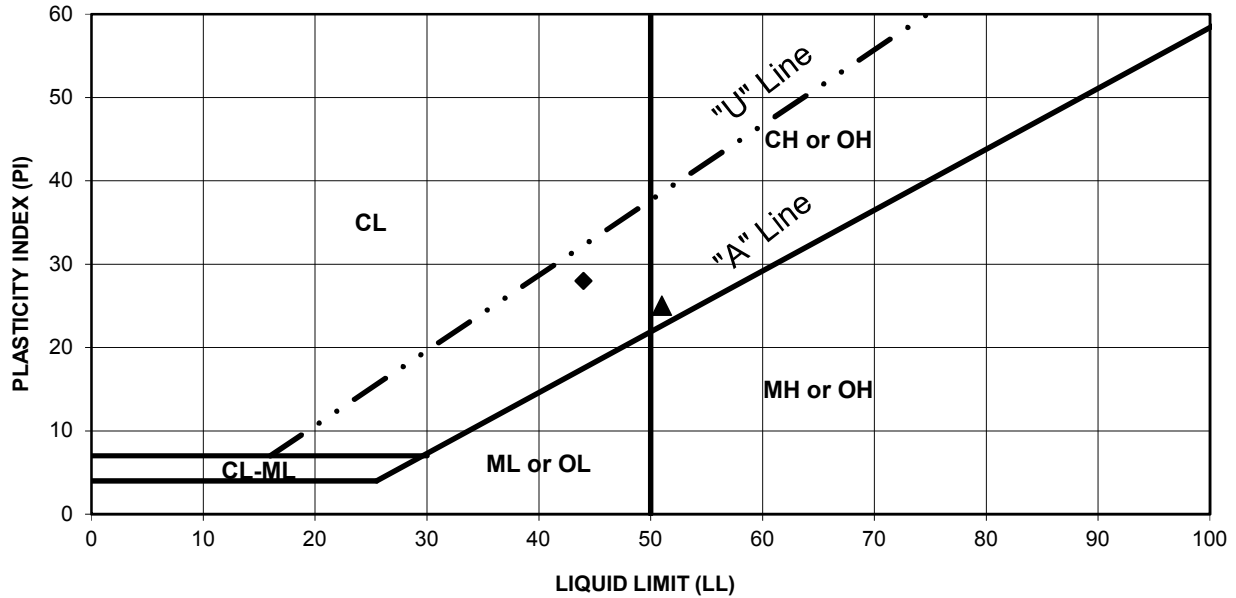


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-1	13a	49	28	15	13	CL
▲	GP-1	14	54	35	16	19	CL



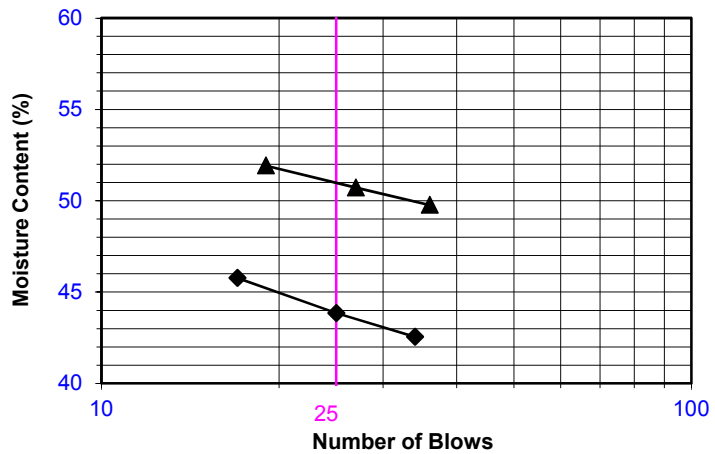
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>NG</u>	Date: <u>09/02/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>09/03/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>09/13/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test

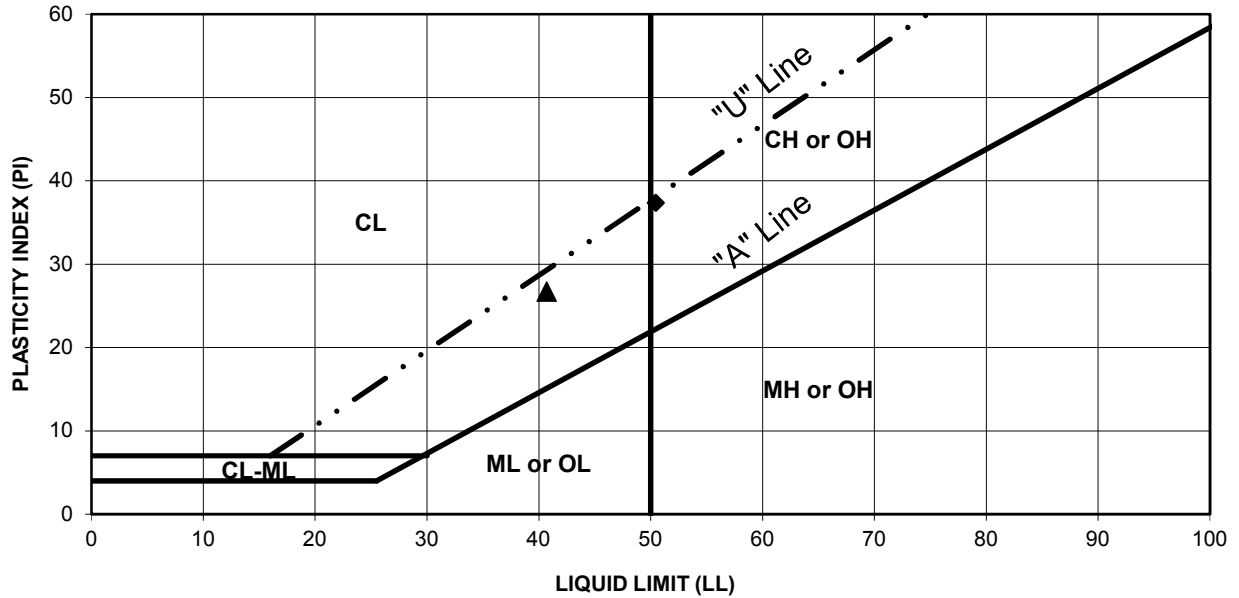


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-1	15a	59	44	16	28	CL
▲	GP-1	19	79	51	26	25	CH



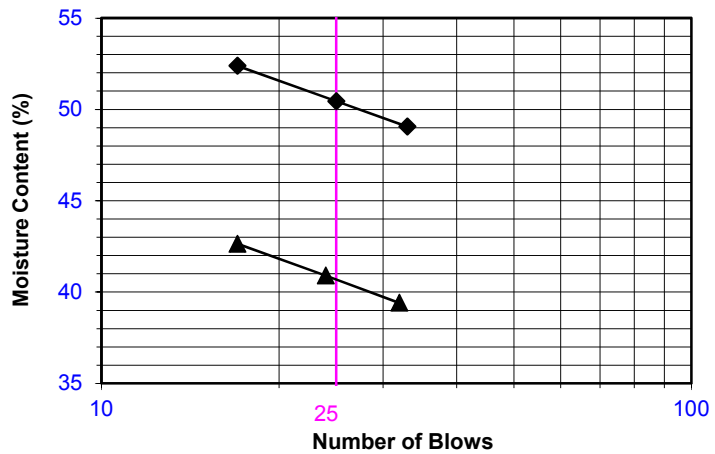
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>LS</u>	Date: <u>09/02/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>09/09/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>09/14/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test

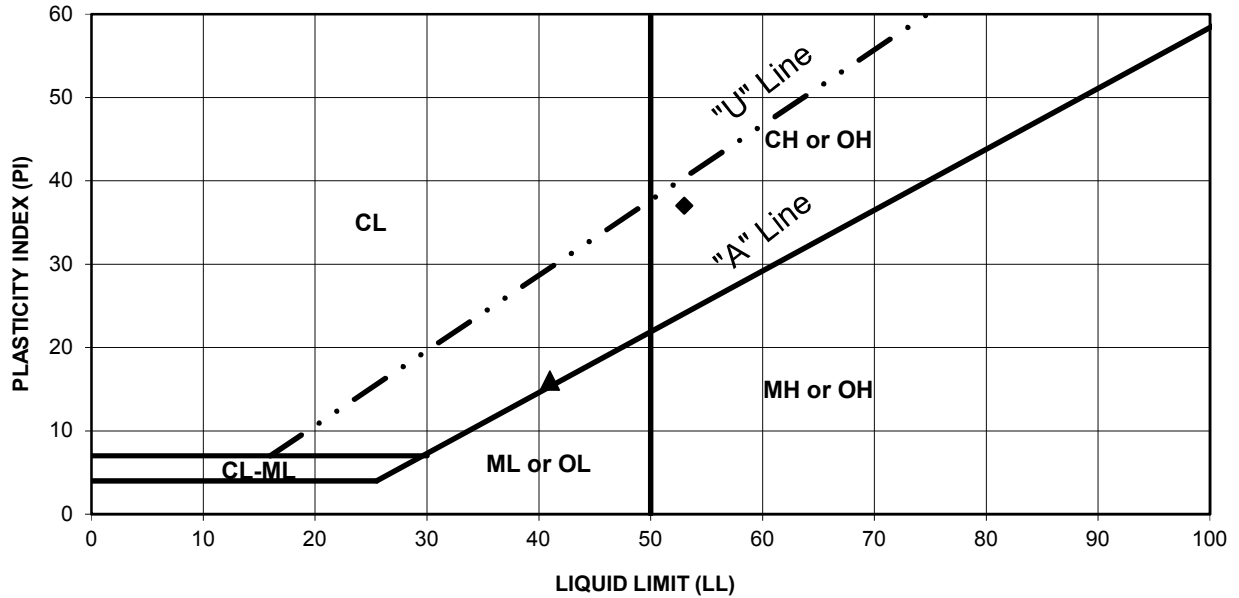


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-2	1a	2.5	50	13	37	CH
▲	GP-2	2	5	41	14	27	CL



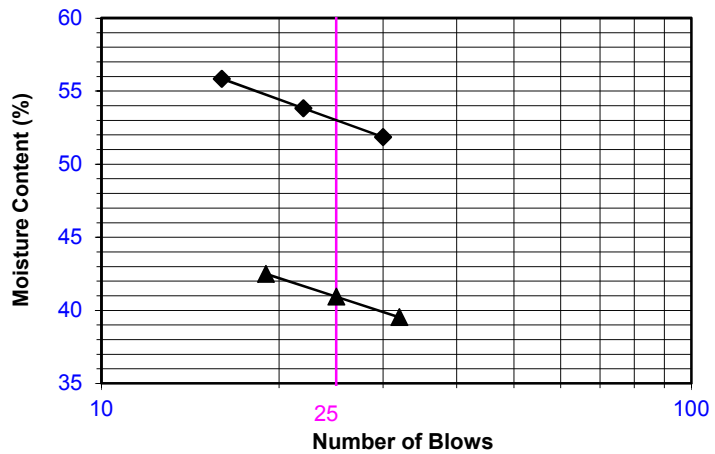
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>LS</u>	Date: <u>09/02/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>09/09/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>09/14/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test

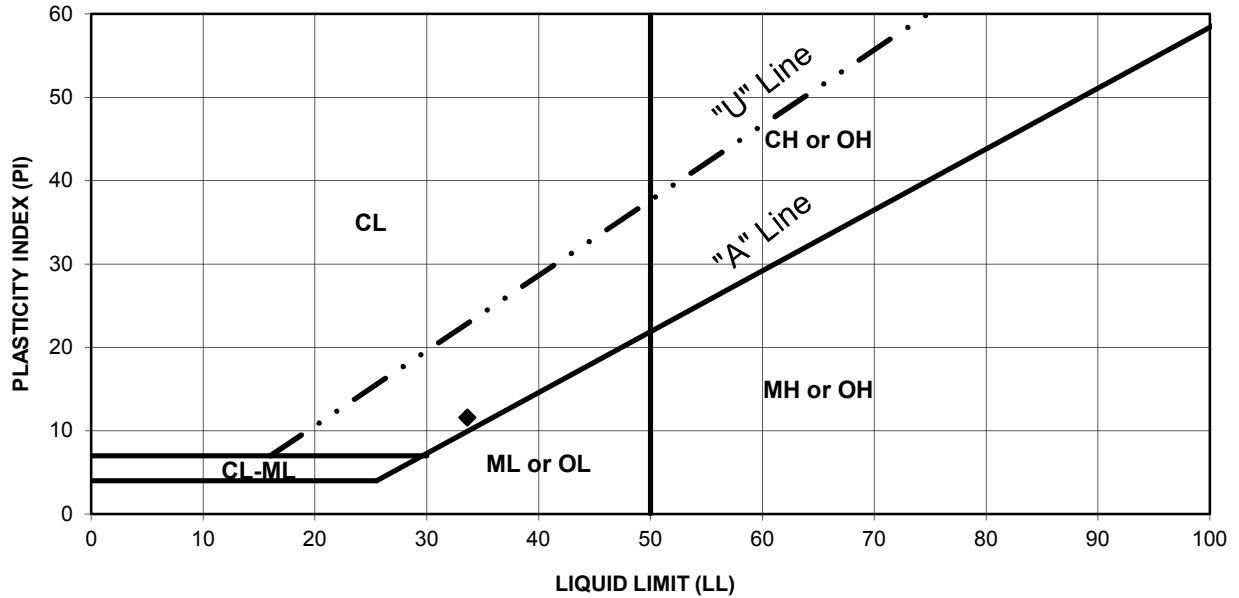


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-2	7a	17	53	16	37	CH
▲	GP-2	9a	24	41	25	16	CL



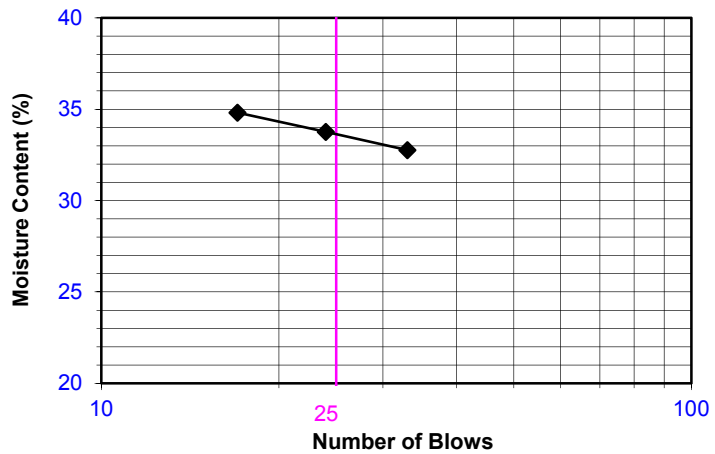
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>LS</u>	Date: <u>09/02/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>09/09/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>09/14/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test



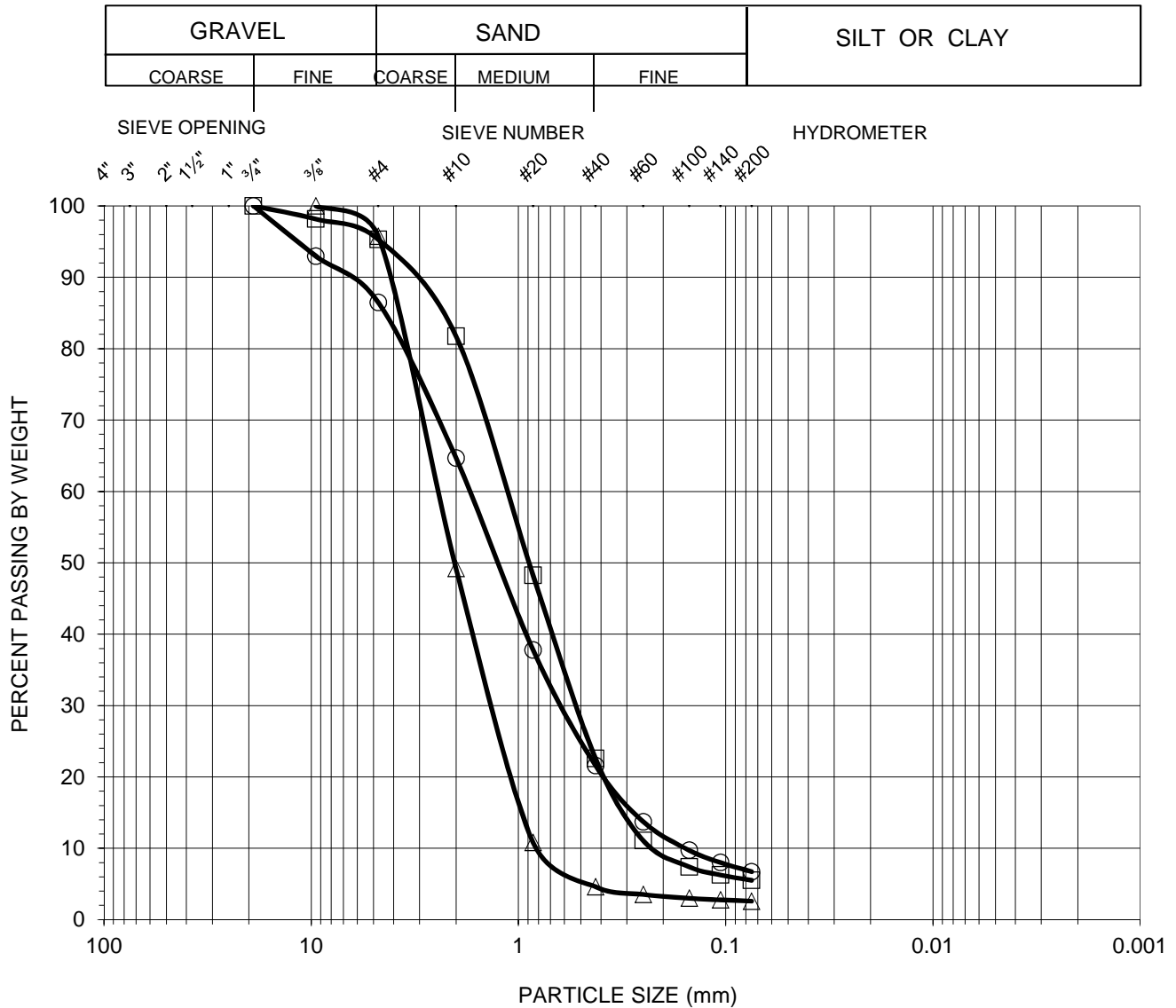
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-2	10	29	34	22	12	CL
	GP-2	11b	34	NP	NP	NP	

* NP denotes "non-plastic"



GRAIN SIZE DISTRIBUTION CURVE ASTM D 6913

Client Name: <u>GeoPentech</u>	Tested by: <u>SM</u>	Date: <u>09/08/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed by: <u>NR</u>	Date: <u>09/09/21</u>
Project No.: <u>21086A</u>	Checked by: <u>AP</u>	Date: <u>09/13/21</u>

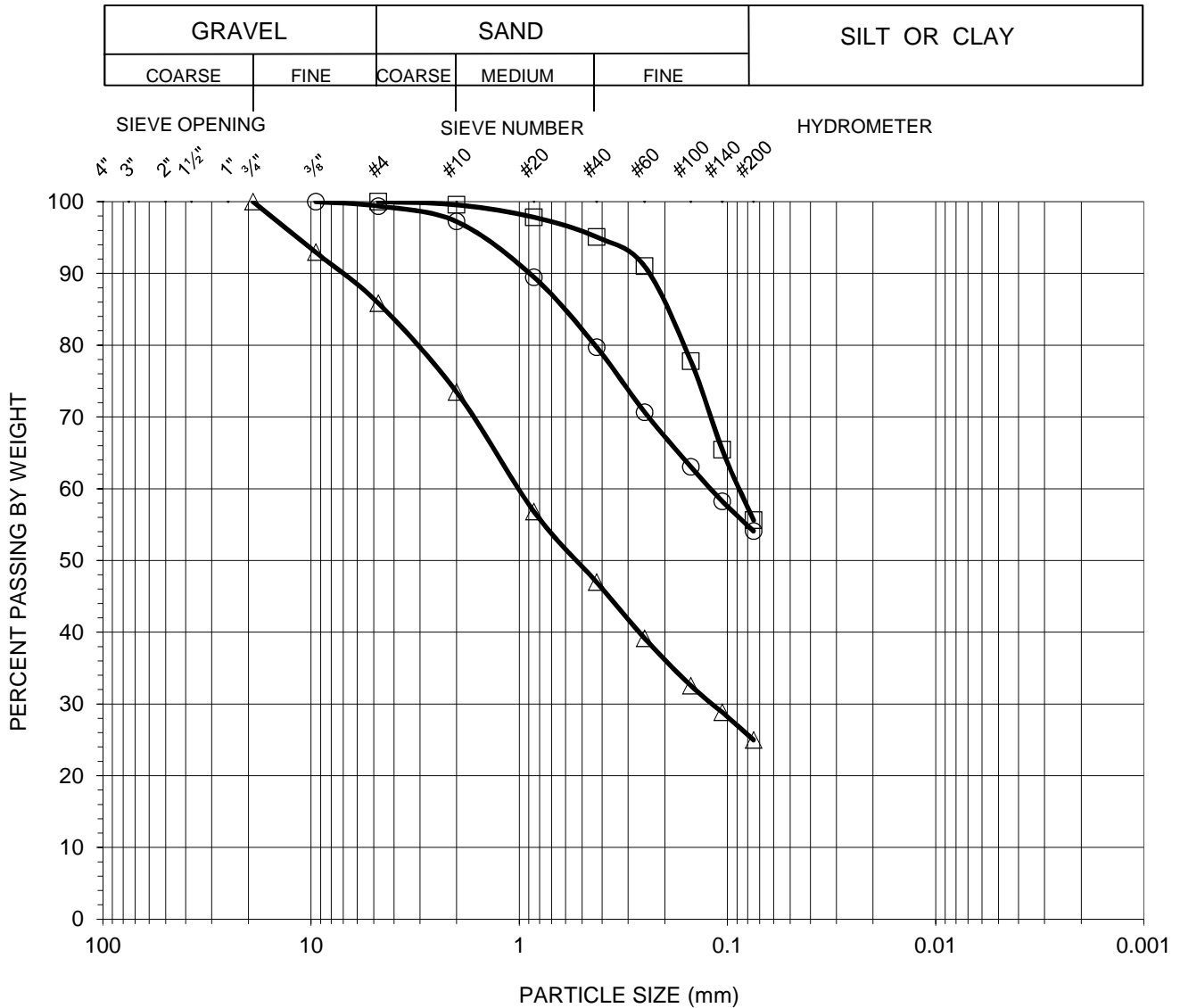


Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	GP-1	2b	5	14	79	7	N/A	SW-SM
□	GP-1	4a	10	5	89	6	N/A	SP-SM
△	GP-1	6a	16	4	93	3	N/A	SP



GRAIN SIZE DISTRIBUTION CURVE ASTM D 6913

Client Name: GeoPentech Tested by: SM Date: 09/08/21
 Project Name: 1056 La Cienega Blvd Computed by: NR Date: 09/09/21
 Project No.: 21086A Checked by: AP Date: 09/13/21



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	GP-2	1a	2.5	1	45	54	50:13:37	CH
□	GP-2	3b	7.5	0	44	56	N/A	CL*
△	GP-2	7a	17	14	61	25	53:16:37	SC

*Note: Based on visual classification of sample

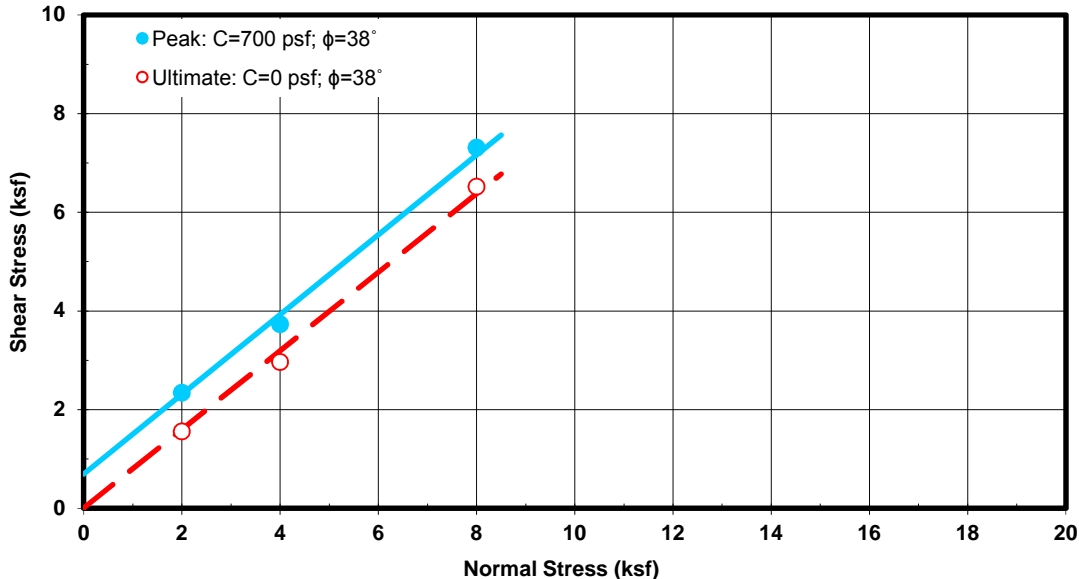
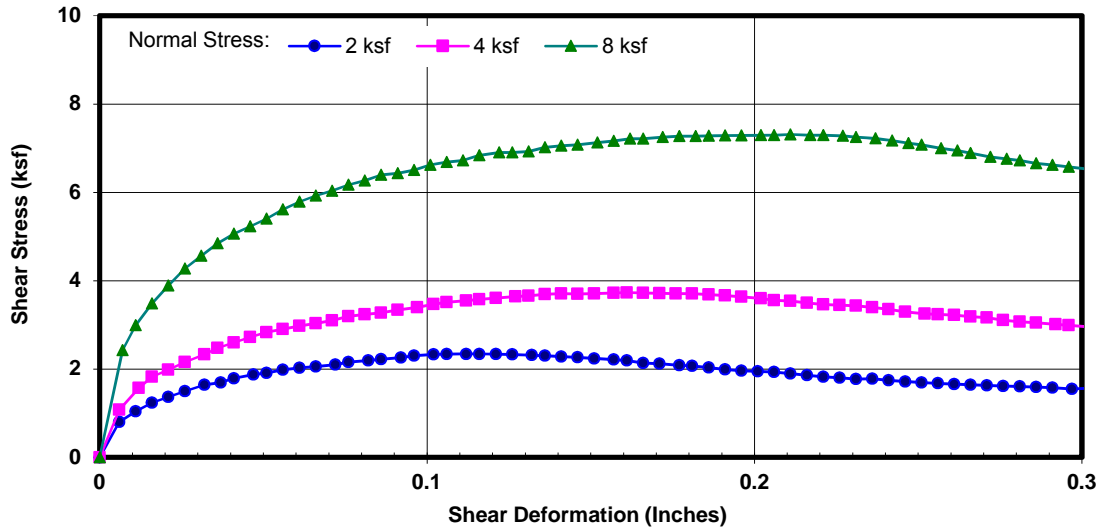


DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 1056 La Cienega Blvd
Project No.: 21086A
Boring No.: GP-1
Sample No.: 6a **Depth (ft):** 16
Sample Type: Mod. Cal.
Soil Description: Poorly Graded Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: SM **Date:** 09/03/21
Computed By: NR **Date:** 09/09/21
Checked by: AP **Date:** 09/13/21

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
110.4	92.6	19.2	26.6	63	88	2	2.341	1.560
						4	3.732	2.964
						8	7.310	6.523



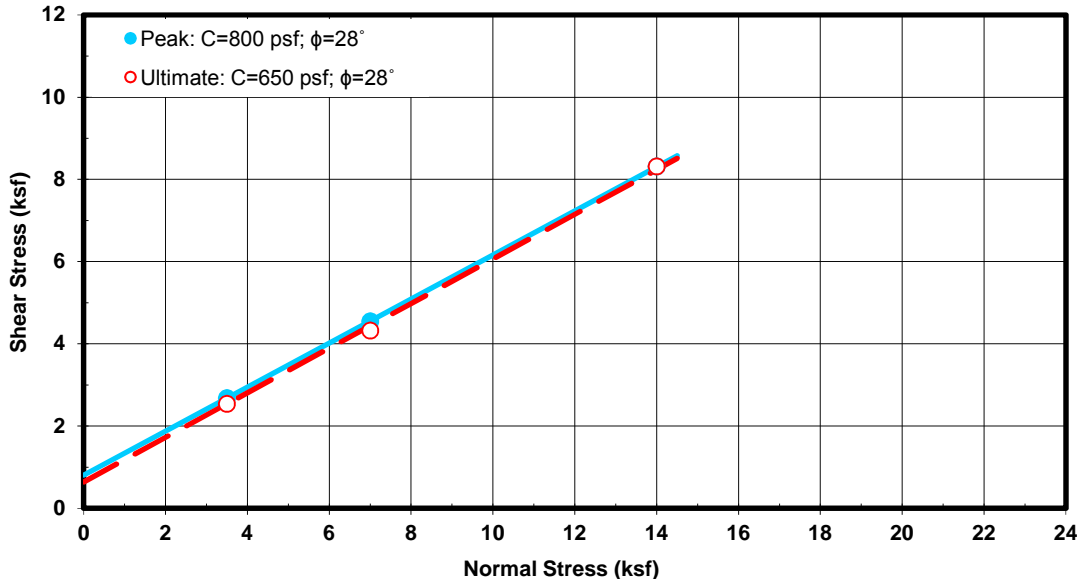
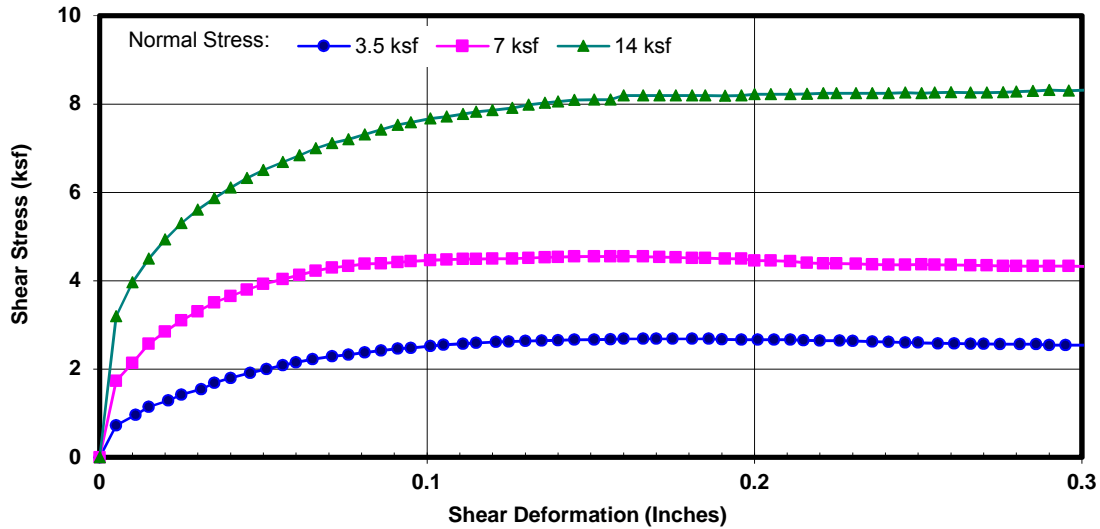


DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 1056 La Cienega Blvd
Project No.: 21086A
Boring No.: GP-1
Sample No.: 9b **Depth (ft):** 29
Sample Type: Mod. Cal.
Soil Description: Sandy Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: SM **Date:** 09/03/21
Computed By: NR **Date:** 09/09/21
Checked by: AP **Date:** 09/13/21

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
130.9	108.9	20.1	20.3	99	100	3.5	2.683	2.540
						7	4.548	4.320
						14	8.315	8.315



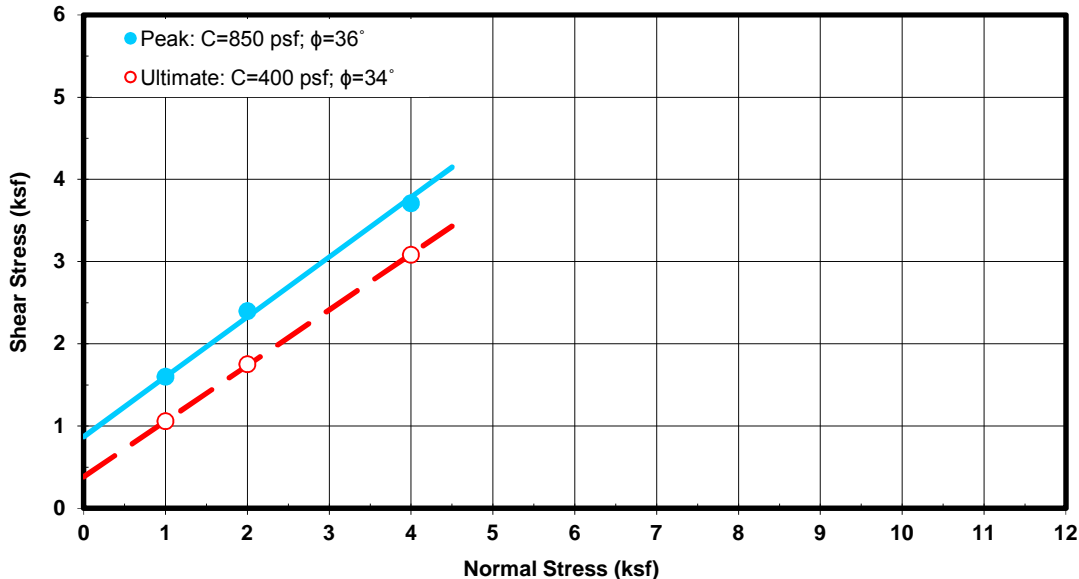
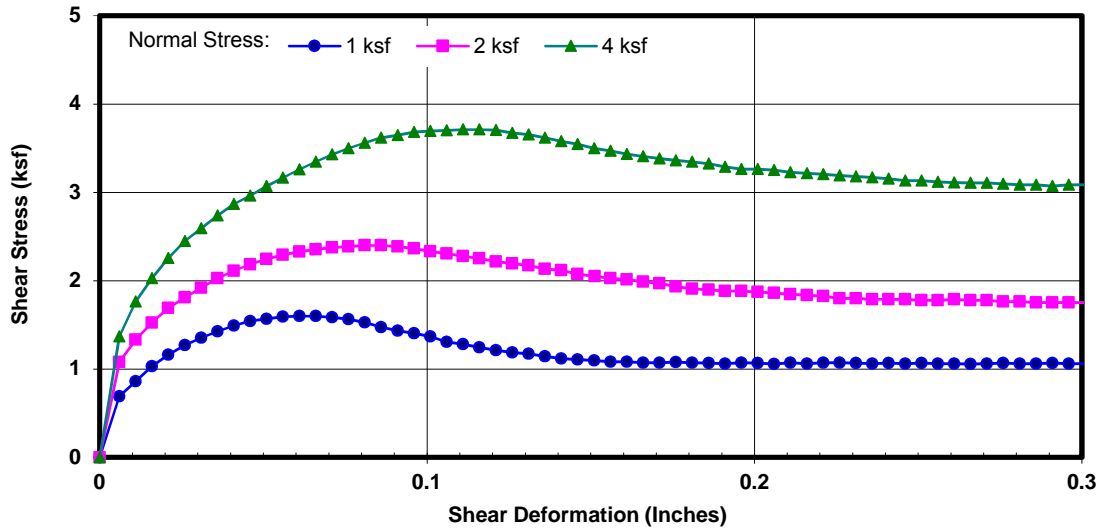


DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 1056 La Cienega Blvd
Project No.: 21086A
Boring No.: GP-2
Sample No.: 3a **Depth (ft):** 7.5
Sample Type: Mod. Cal.
Soil Description: Sandy Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: SM **Date:** 09/03/21
Computed By: NR **Date:** 09/09/21
Checked by: AP **Date:** 09/13/21

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.0	109.4	17.9	19.9	90	100	1	1.599	1.061
						2	2.400	1.752
						4	3.711	3.084



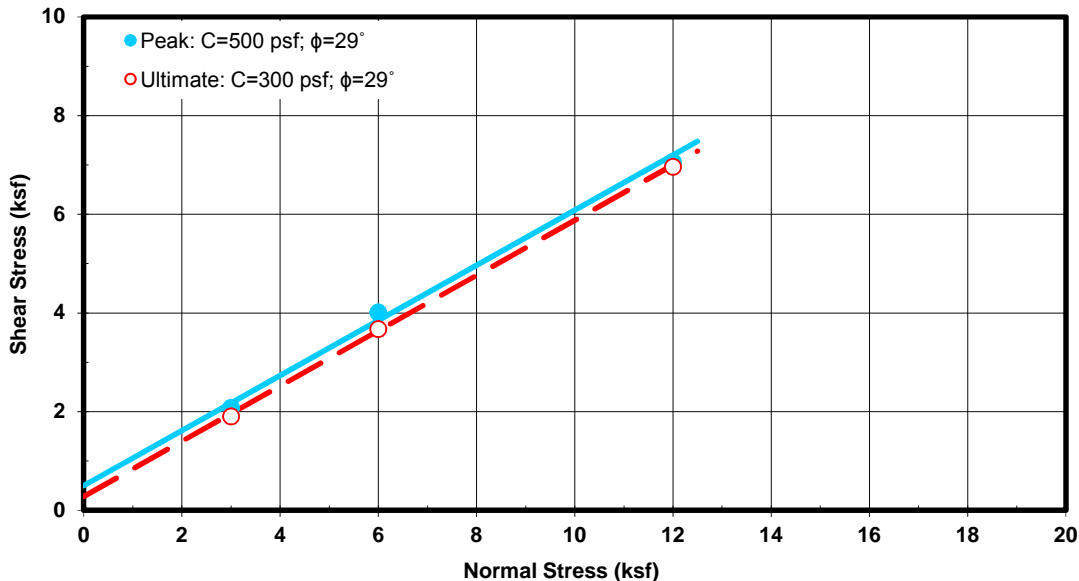
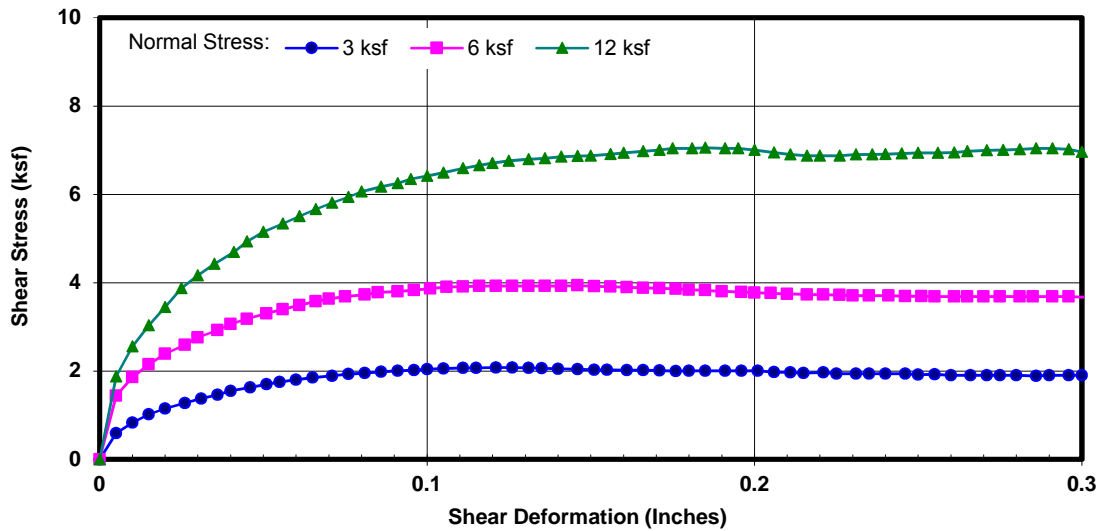


DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 1056 La Cienega Blvd
Project No.: 21086A
Boring No.: GP-2
Sample No.: 9a **Depth (ft):** 24
Sample Type: Mod. Cal.
Soil Description: Lean Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: SM **Date:** 09/07/21
Computed By: NR **Date:** 09/09/21
Checked by: AP **Date:** 09/14/21

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
120.3	92.2	30.5	30.7	99	100	3	2.079	1.903
						6	4.008	3.672
						12	7.056	6.960



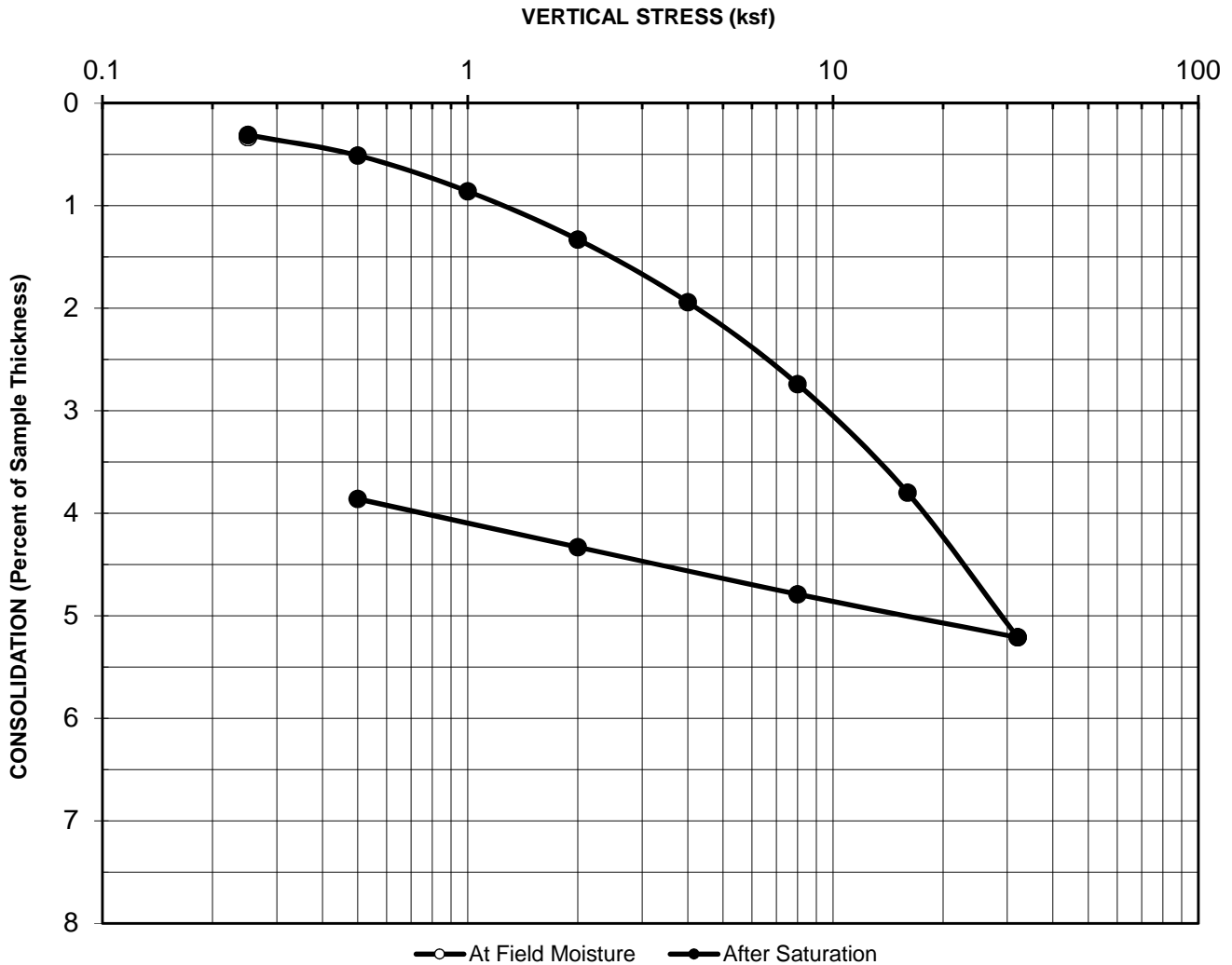


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Boring No. : GP-1

Initial Dry Unit Weight (pcf): 113.6

Sample No.: 9a

Initial Moisture Content (%): 17.2

Depth (feet): 29

Final Moisture Content (%): 17.4

Sample Type: Mod Cal

Assumed Specific Gravity: 2.7

Soil Description: Clayey Sand w/gravel

Initial Void Ratio: 0.48

Remarks: Swell= 0.02% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 9/1/2021

AP No: 21-0873 Sheet No: 1

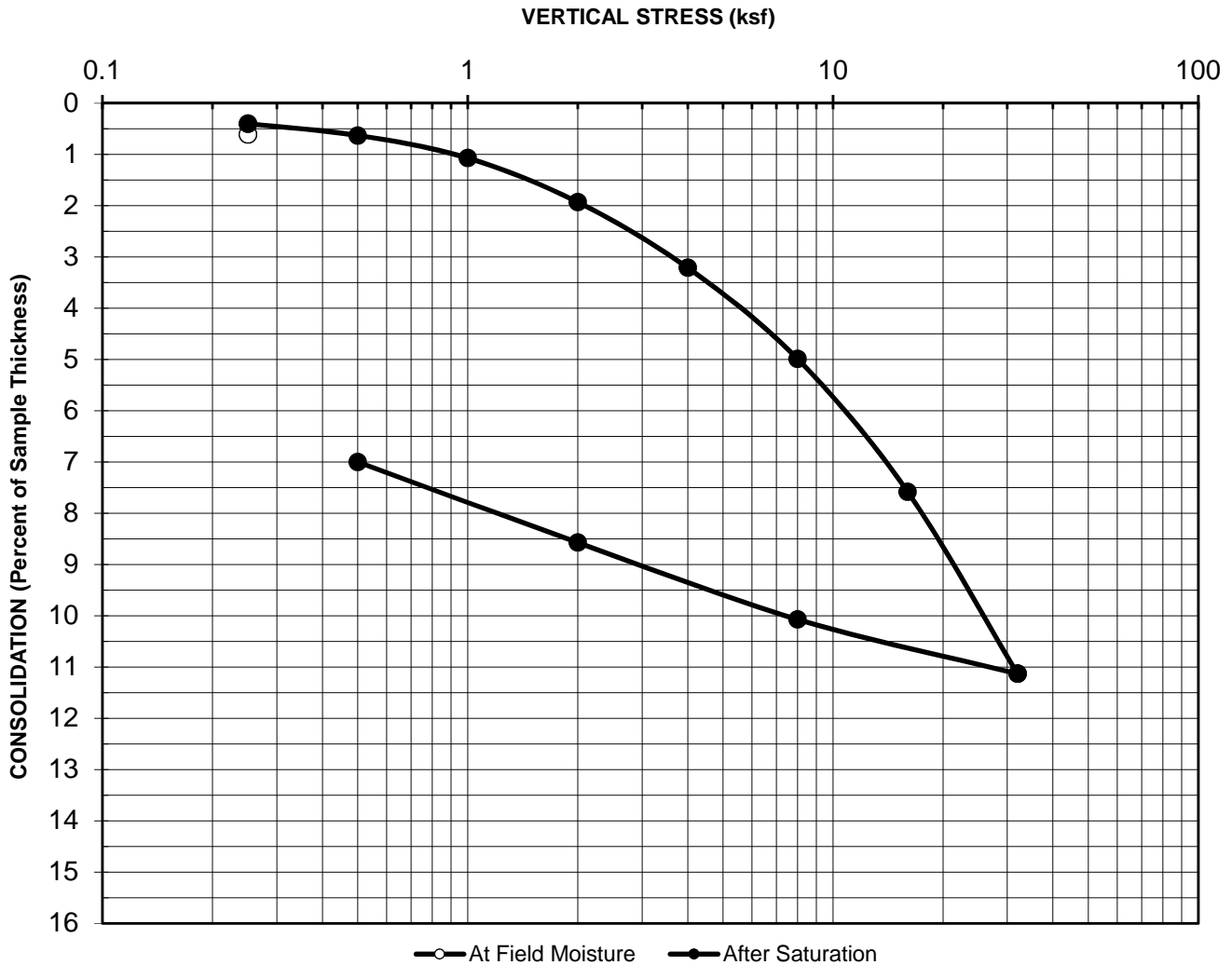


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Boring No. : GP-1

Initial Dry Unit Weight (pcf): 95.1

Sample No.: 13a

Initial Moisture Content (%): 27.9

Depth (feet): 49

Final Moisture Content (%): 28.1

Sample Type: Mod Cal

Assumed Specific Gravity: 2.7

Soil Description: Lean Clay

Initial Void Ratio: 0.77

Remarks: Swell= 0.21% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 9/1/2021

AP No: 21-0873 Sheet No: 1

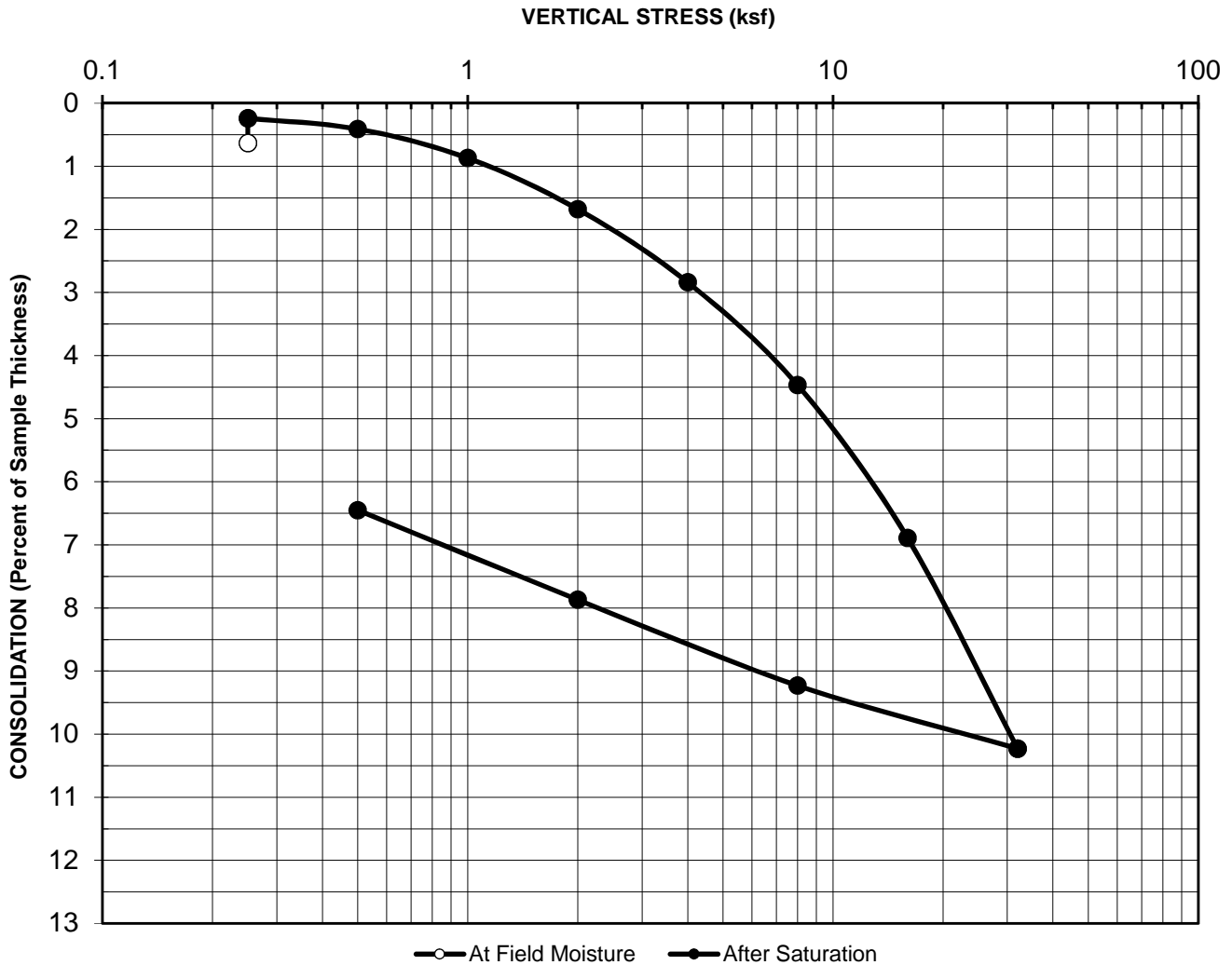


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Boring No. : GP-1

Initial Dry Unit Weight (pcf): 102.9

Sample No.: 15b

Initial Moisture Content (%): 22.9

Depth (feet): 59

Final Moisture Content (%): 23.0

Sample Type: Mod Cal

Assumed Specific Gravity: 2.7

Soil Description: Sandy Clay

Initial Void Ratio: 0.64

Remarks: Swell= 0.39% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 9/1/2021

AP No: 21-0873 Sheet No: 1

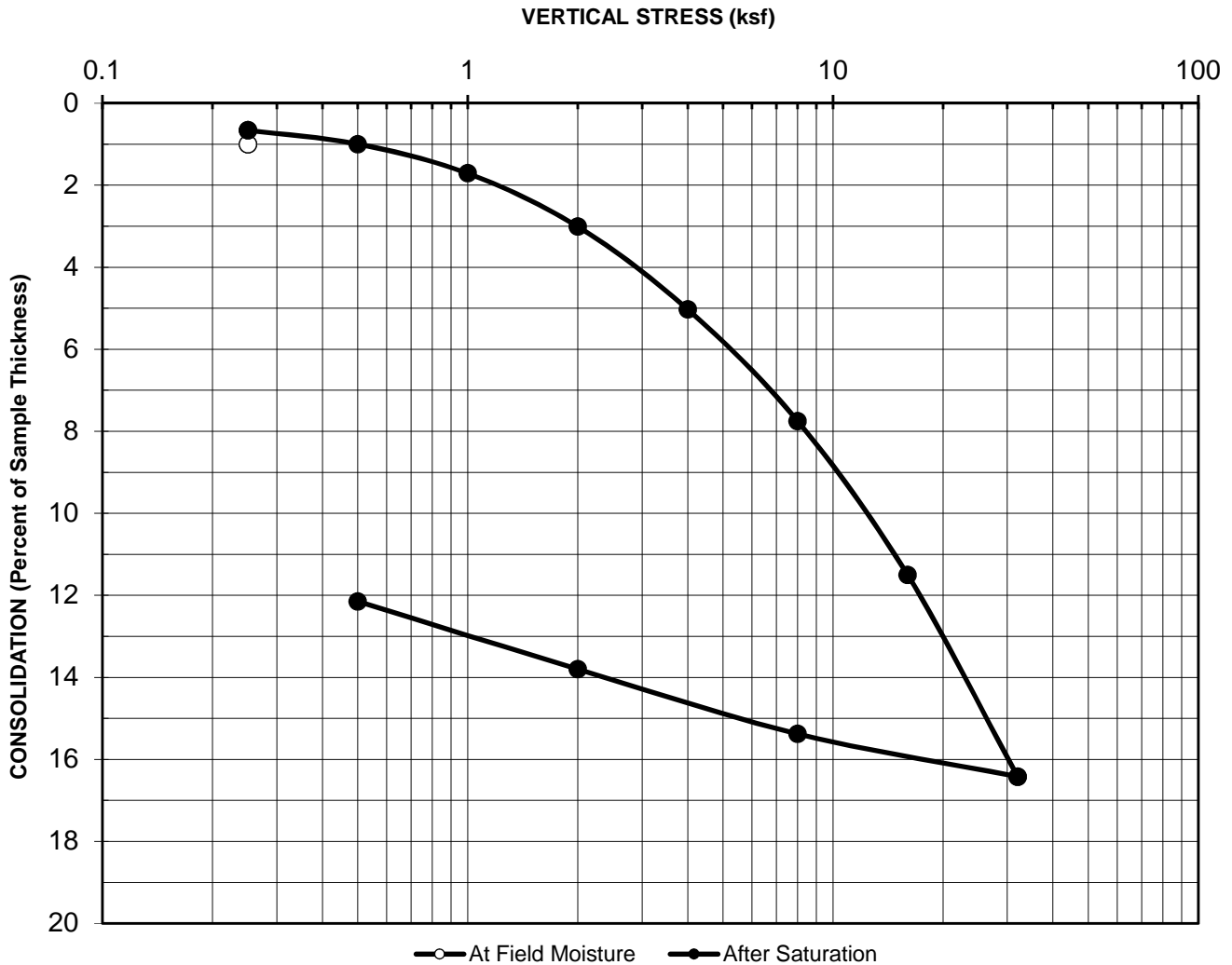


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Boring No. : GP-2

Initial Dry Unit Weight (pcf): 89.9

Sample No.: 9a

Initial Moisture Content (%): 31.3

Depth (feet): 24

Final Moisture Content (%): 31.4

Sample Type: Mod Cal

Assumed Specific Gravity: 2.7

Soil Description: Lean Clay

Initial Void Ratio: 0.87

Remarks: Swell= 0.34% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 9/1/2021

AP No: 21-0873 **Sheet No:** 1

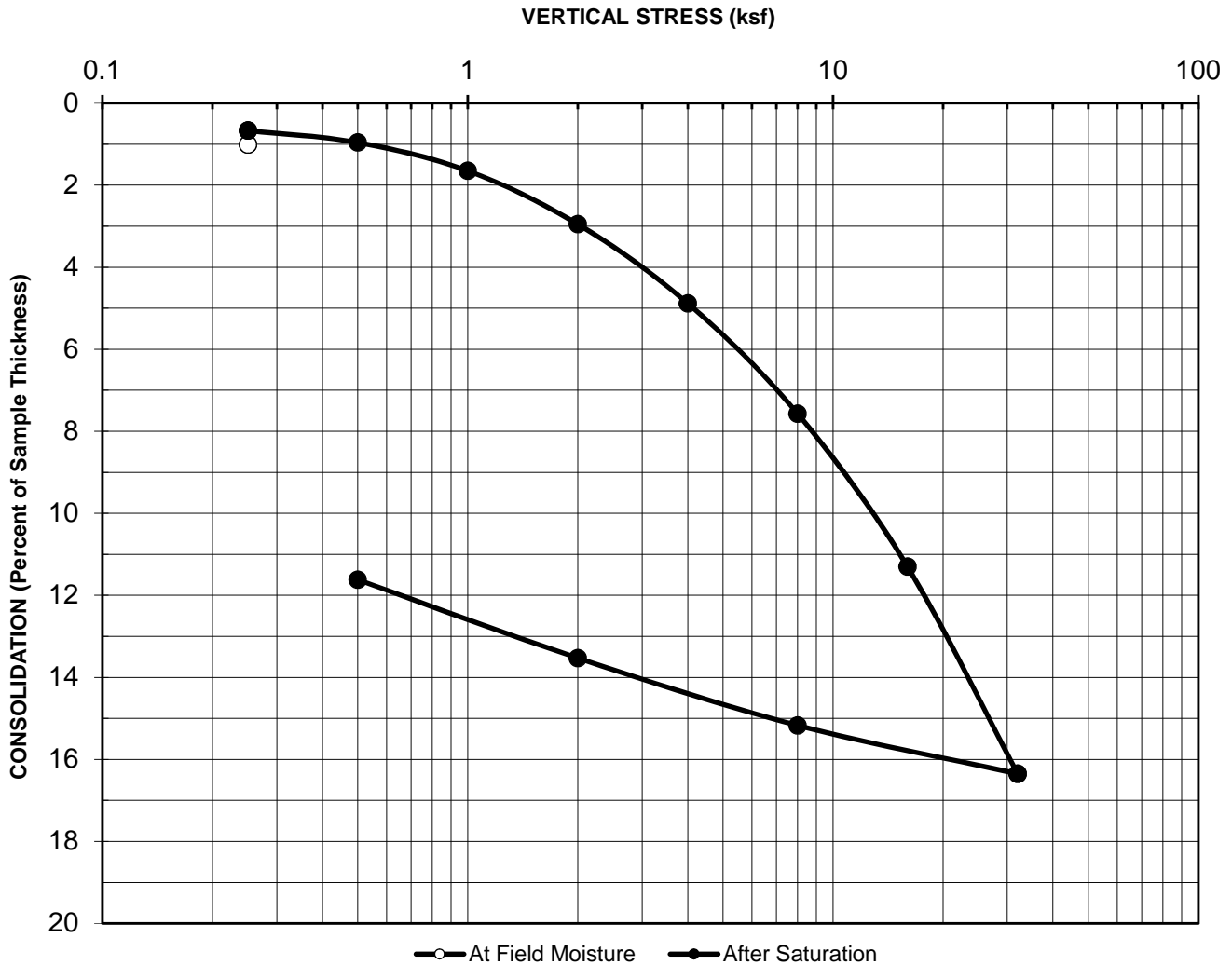


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Boring No. : GP-2

Initial Dry Unit Weight (pcf): 91.4

Sample No.: 13a

Initial Moisture Content (%): 31.1

Depth (feet): 44

Final Moisture Content (%): 31.1

Sample Type: Mod Cal

Assumed Specific Gravity: 2.7

Soil Description: Lean Clay

Initial Void Ratio: 0.84

Remarks: Swell= 0.34% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 9/1/2021

AP No: 21-0873 **Sheet No:** 1



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CORROSION TEST RESULTS

Client Name: GeoPentech
Project Name: 1056 La Cienega Blvd
Project No.: 21086A

AP Job No.: 21-0873
Date: 09/03/21

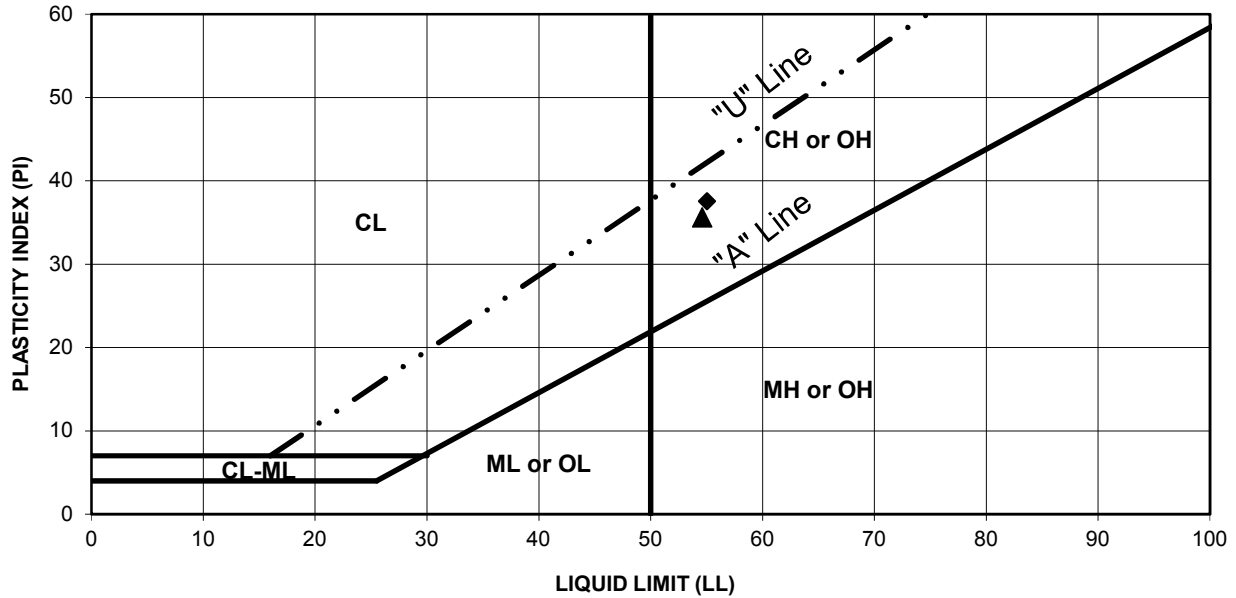
Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
GP-1	Bulk-1	0-5	Clayey Sand w/gravel	3,863	8.3	62	56

NOTES: Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested



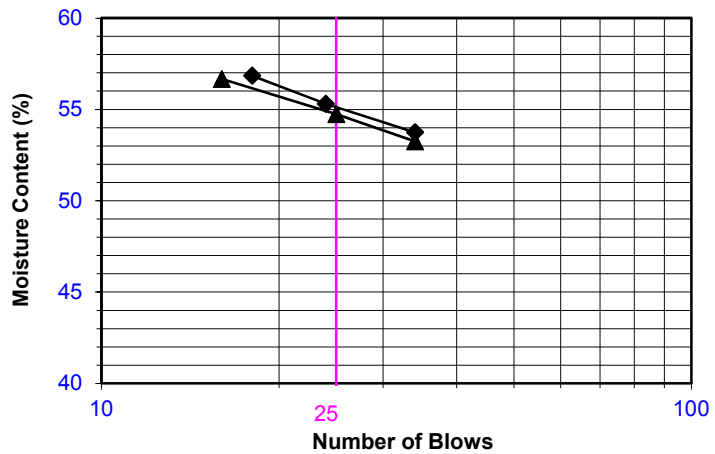
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>DK</u>	Date: <u>11/09/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>11/10/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>11/15/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test

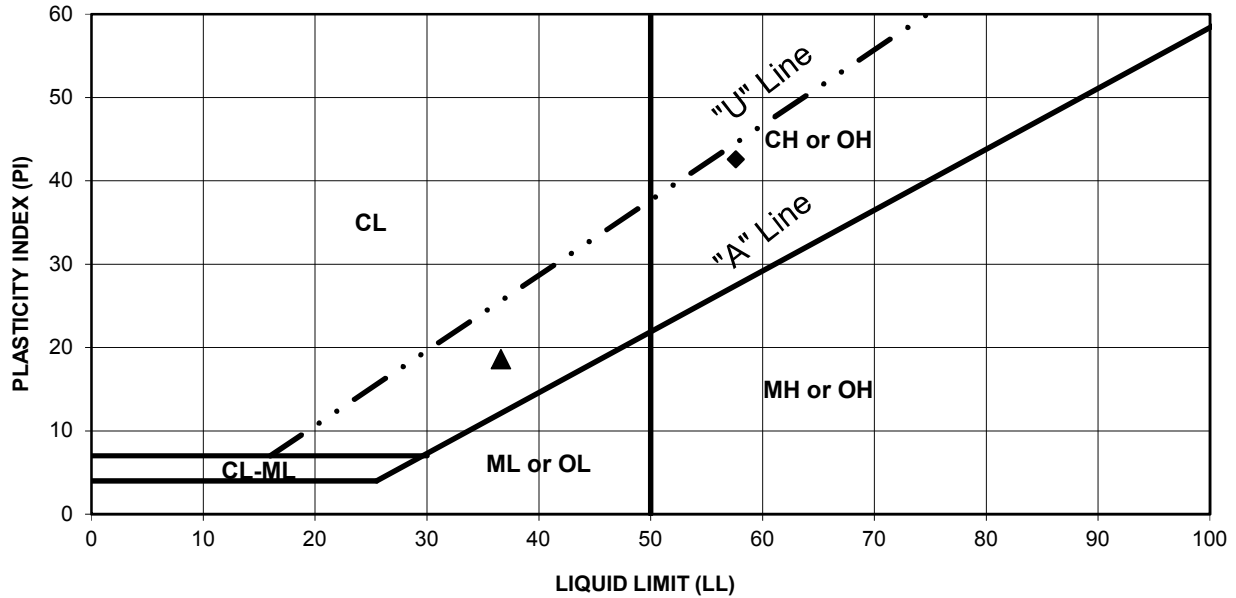


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-3	2a	10	55	17	38	CH
▲	GP-3	4a	19	55	19	36	CH



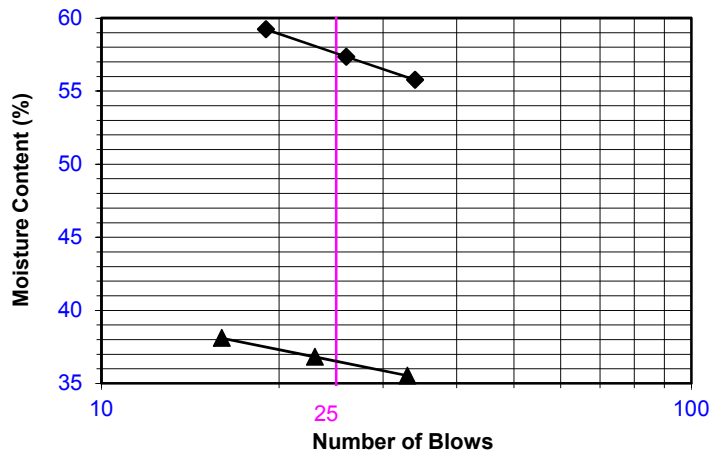
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>DK</u>	Date: <u>11/09/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>11/10/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>11/15/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test

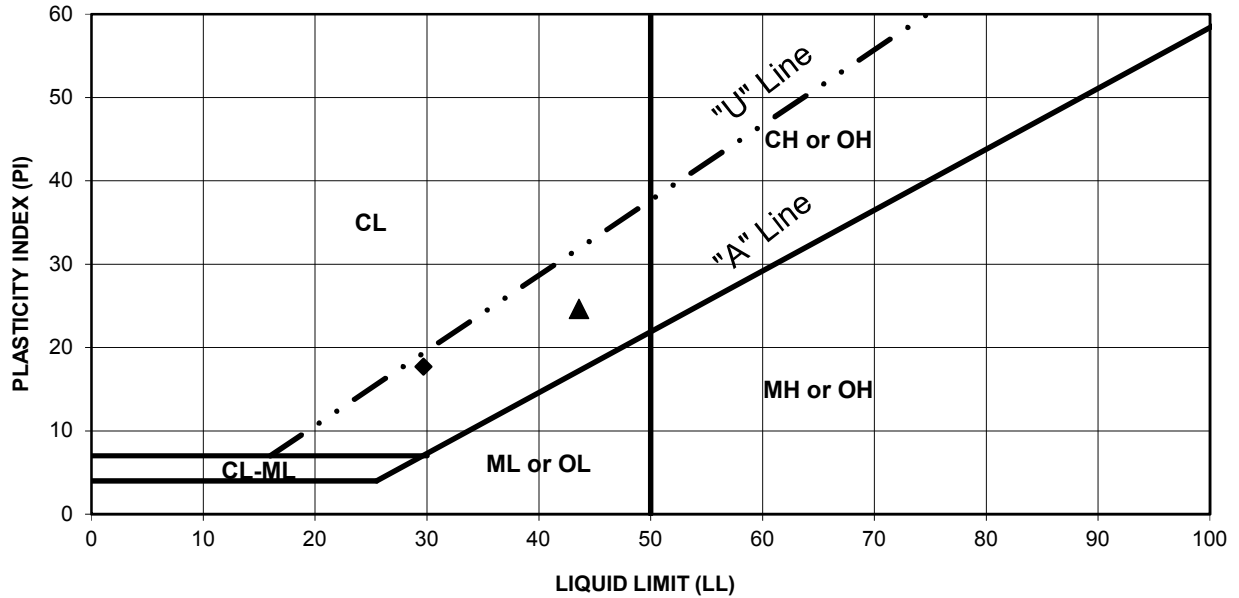


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-3	6	29	58	15	43	CH
▲	GP-3	11	45.5	37	18	19	CL



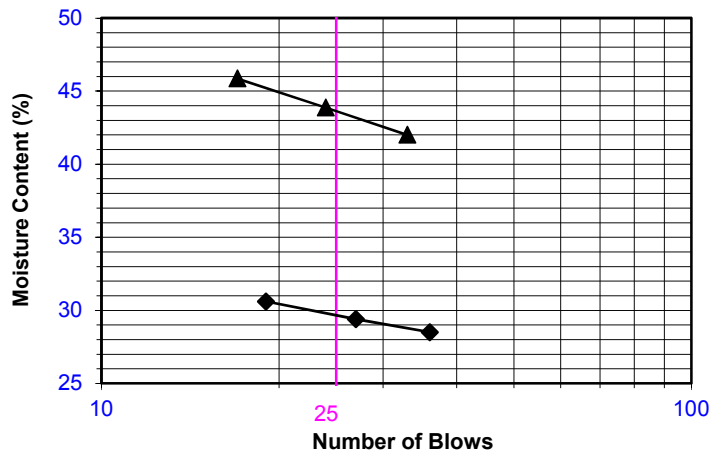
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>DK</u>	Date: <u>11/09/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>11/10/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>11/15/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test

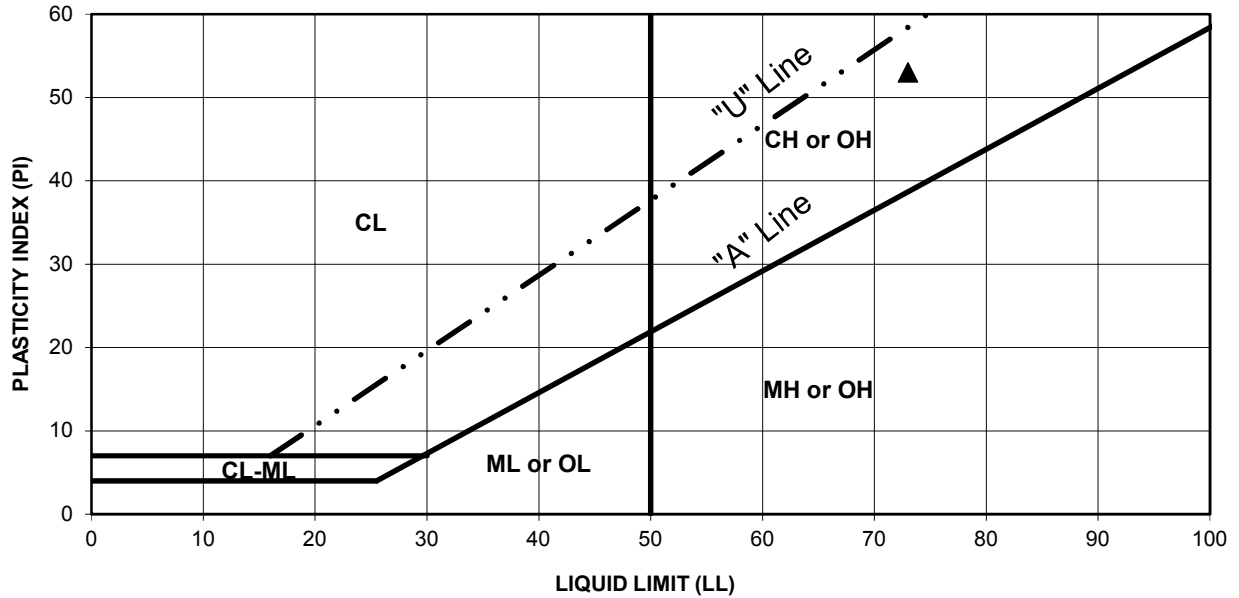


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-3	13a	54	30	12	18	CL
▲	GP-3	14	55.5	44	19	25	CL



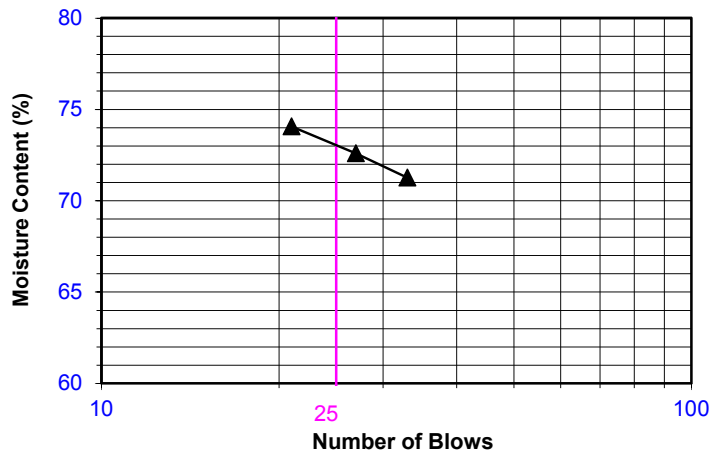
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>DK</u>	Date: <u>11/09/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>11/10/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>11/15/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test



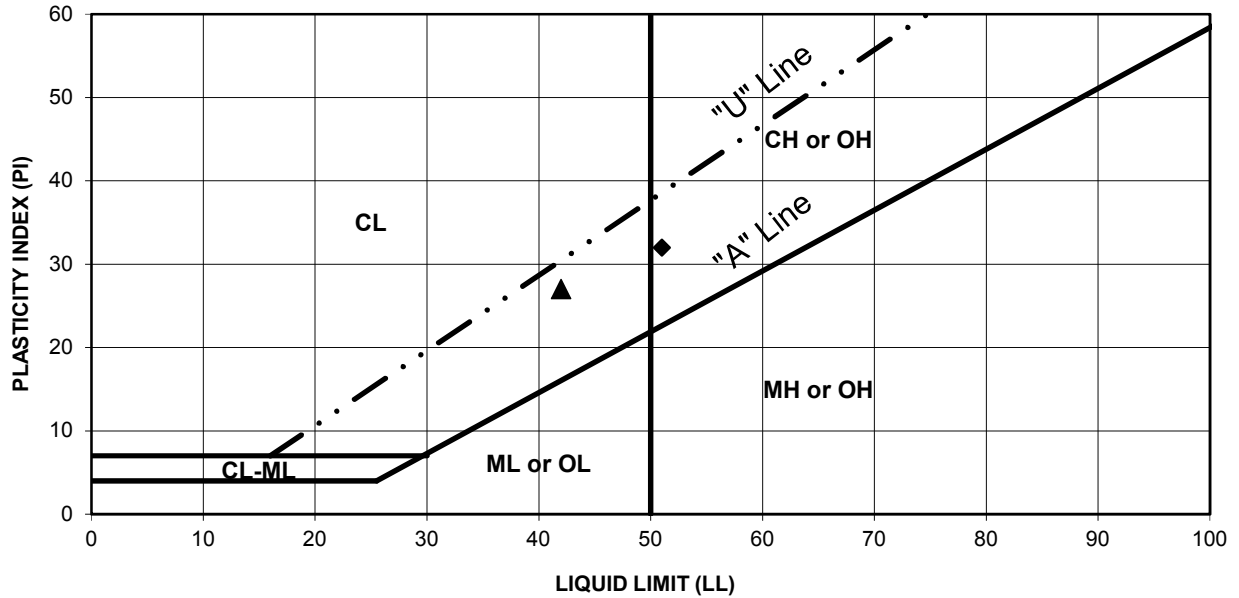
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	GP-3	19a	79	NP	NP	NP	
▲	GP-3	22a	99	73	20	53	CH

* NP denotes "non-plastic"



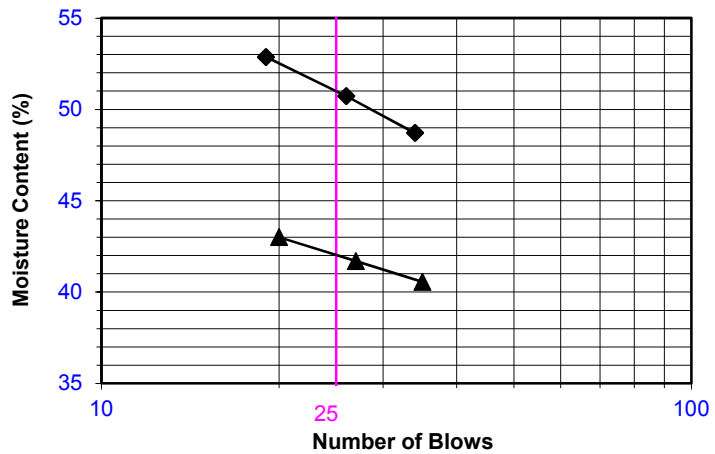
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>DK</u>	Date: <u>11/09/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>11/10/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>11/15/21</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
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- Procedure B
One-point Test

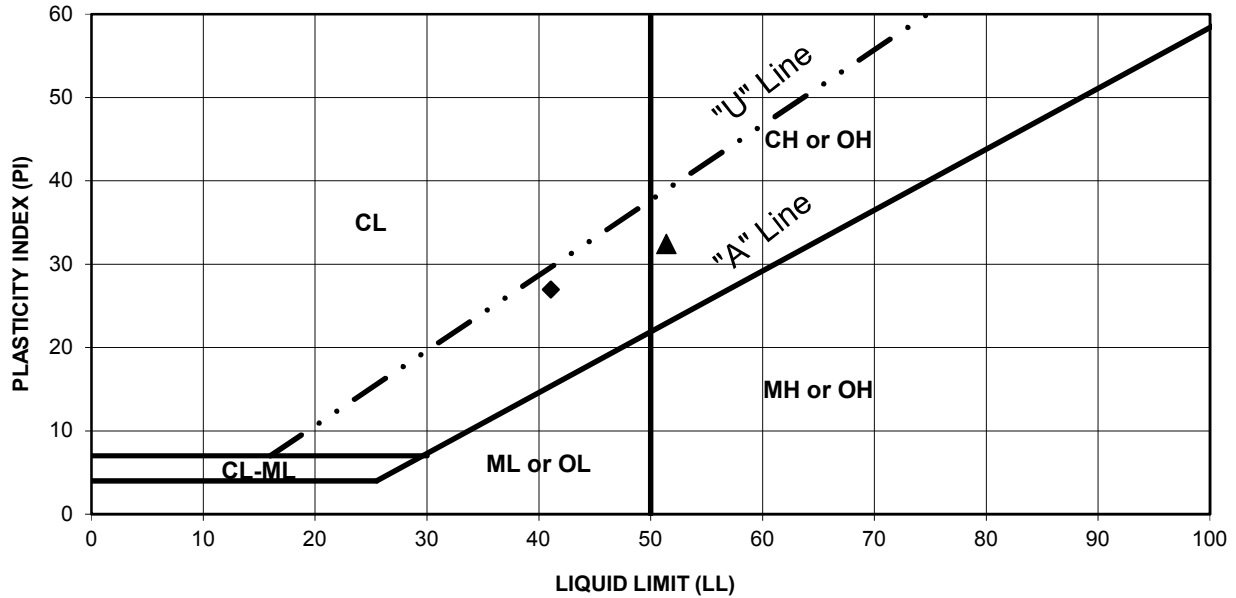


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-3	23-top	104.5	51	19	32	CH
▲	GP-3	24a	109	42	15	27	CL



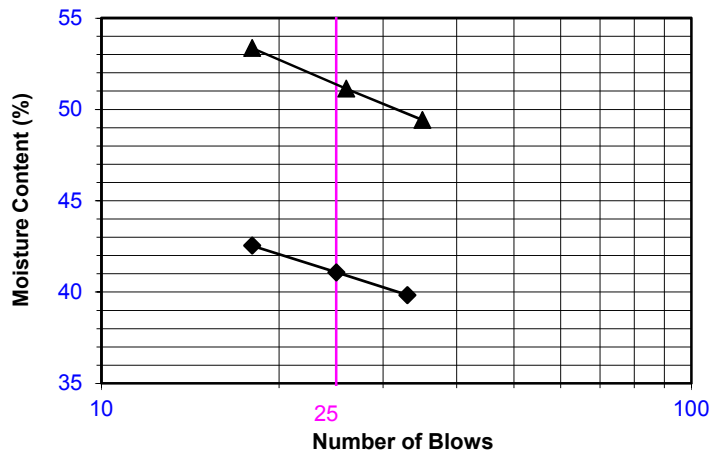
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>DK</u>	Date: <u>11/09/21</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>11/10/21</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>11/15/21</u>



PROCEDURE USED

- Wet Preparation
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Multipoint Test
- Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-3	27a	129	41	14	27	CL
▲	GP-3	28a	139	51	19	32	CH

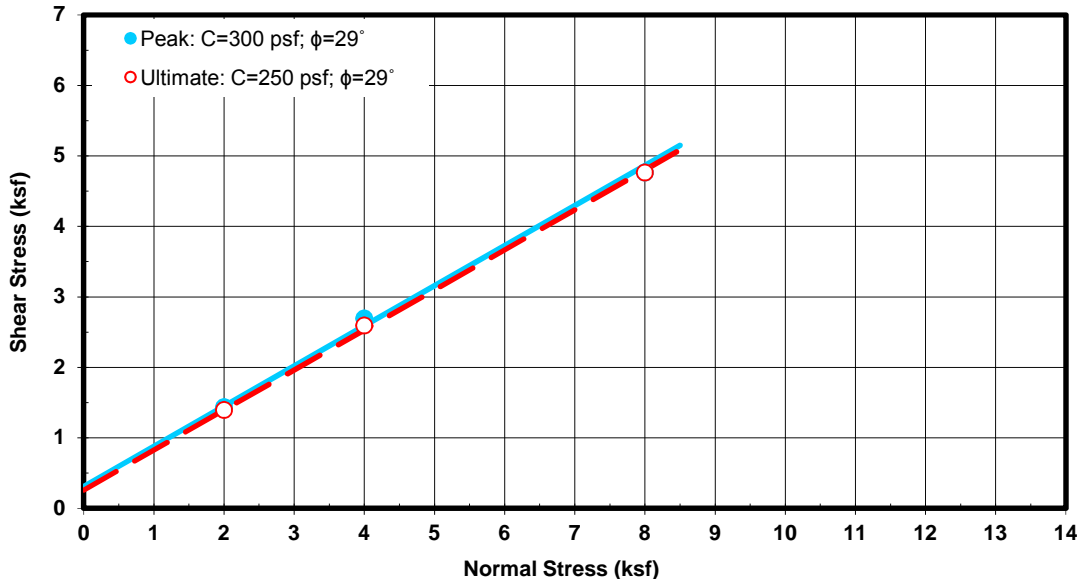
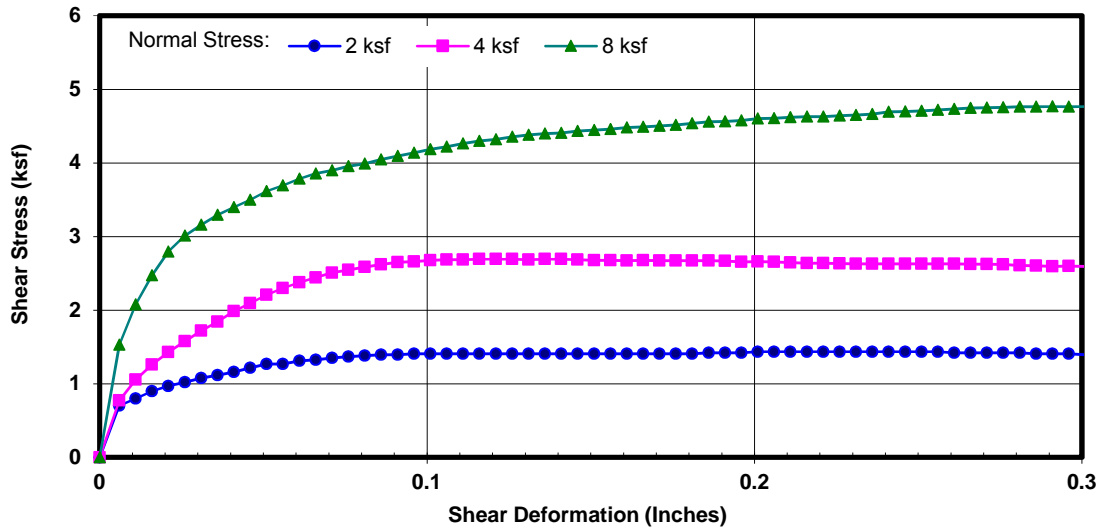


DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 1056 La Cienega Blvd
Project No.: 21086A
Boring No.: GP-3
Sample No.: 4b **Depth (ft):** 19
Sample Type: Mod. Cal.
Soil Description: Sandy Clay w/gravel
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 11/09/21
Computed By: NR **Date:** 11/10/21
Checked by: AP **Date:** 11/15/21

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
114.8	94.1	22.0	29.4	75	100	2	1.435	1.394
						4	2.692	2.592
						8	4.766	4.766



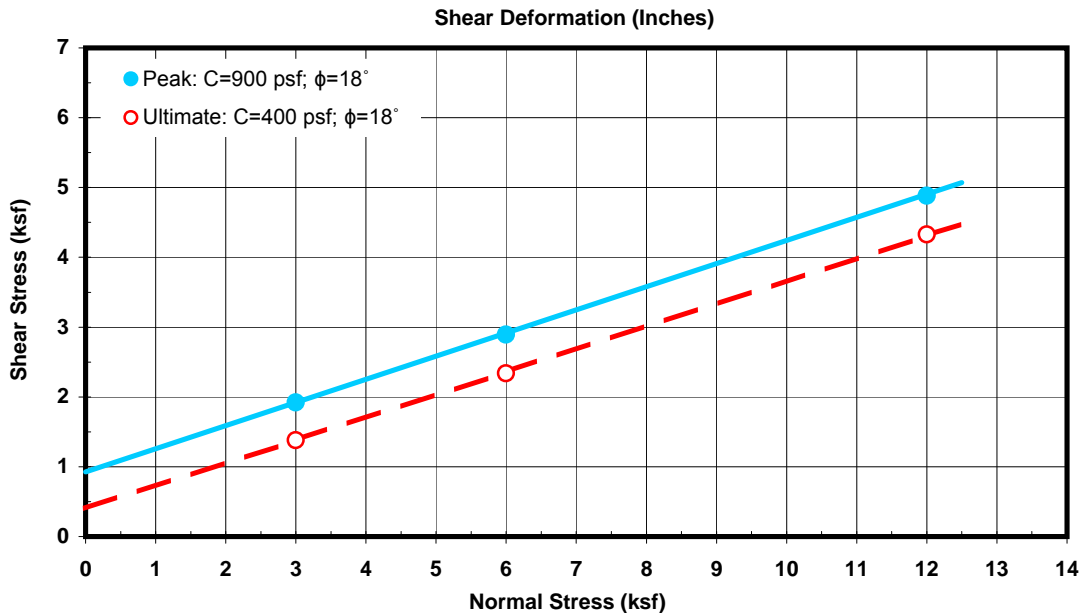
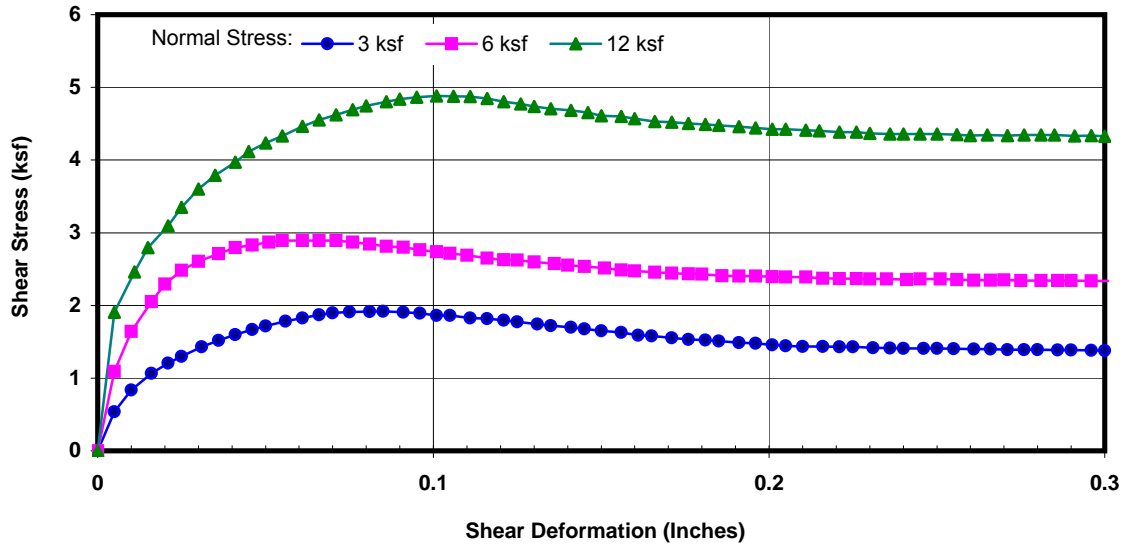


DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 1056 La Cienega Blvd
Project No.: 21086A
Boring No.: GP-3
Sample No.: 5b **Depth (ft):** 24
Sample Type: Mod. Cal.
Soil Description: Fat Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 11/09/21
Computed By: NR **Date:** 11/10/21
Checked by: AP **Date:** 11/15/21

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
115.5	88.3	30.9	33.7	92	100	3	1.920	1.380
						6	2.892	2.336
						12	4.882	4.328



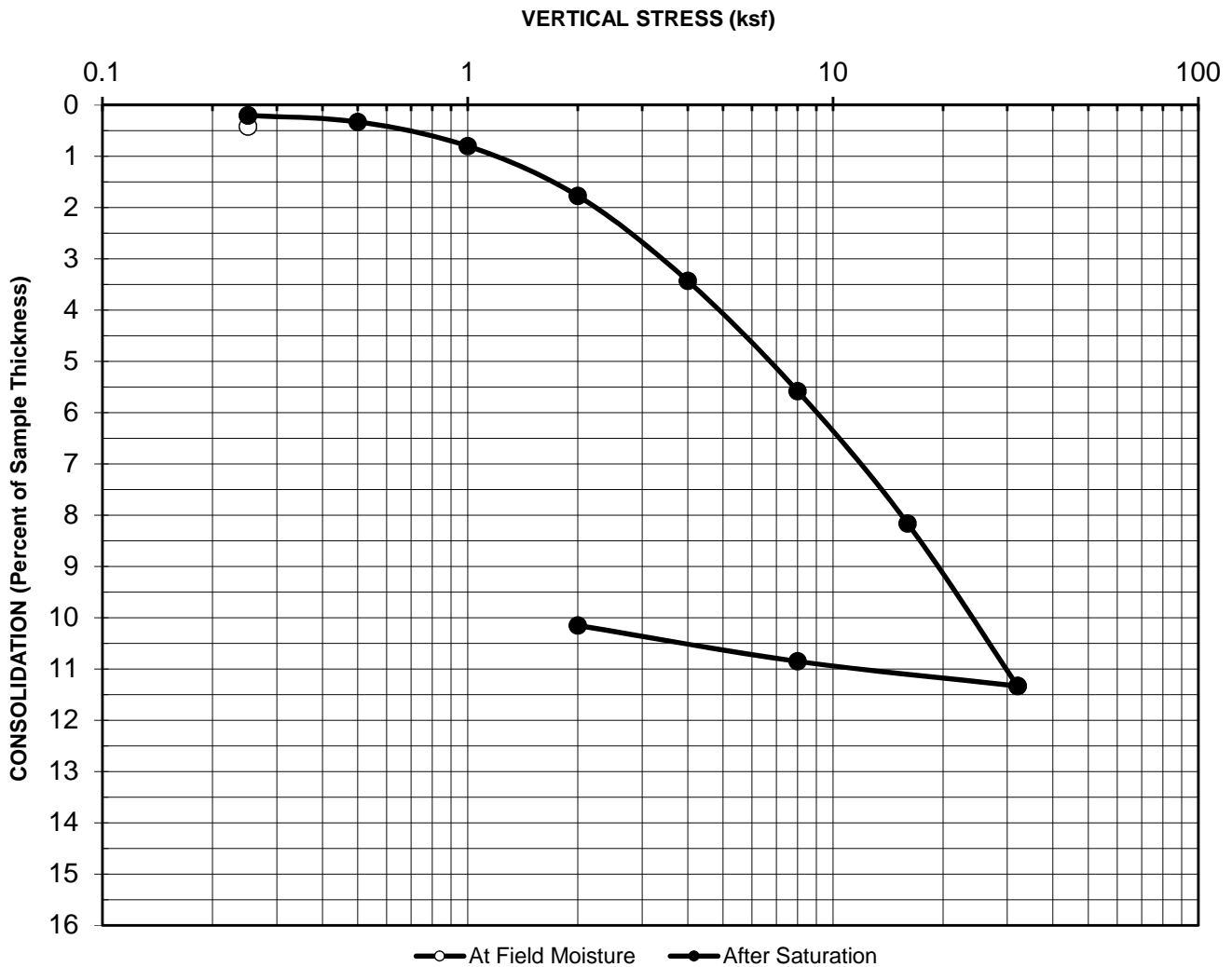


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Boring No. : GP-3

Initial Dry Unit Weight (pcf): 105.8

Sample No.: 13a

Initial Moisture Content (%): 21.6

Depth (feet): 54

Final Moisture Content (%): 21.5

Sample Type: Mod Cal

Assumed Specific Gravity: 2.7

Soil Description: Sandy Lean Clay

Initial Void Ratio: 0.59

Remarks: Swell= 0.22% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 11/8/2021

AP No: 21-0873 Sheet No: 1

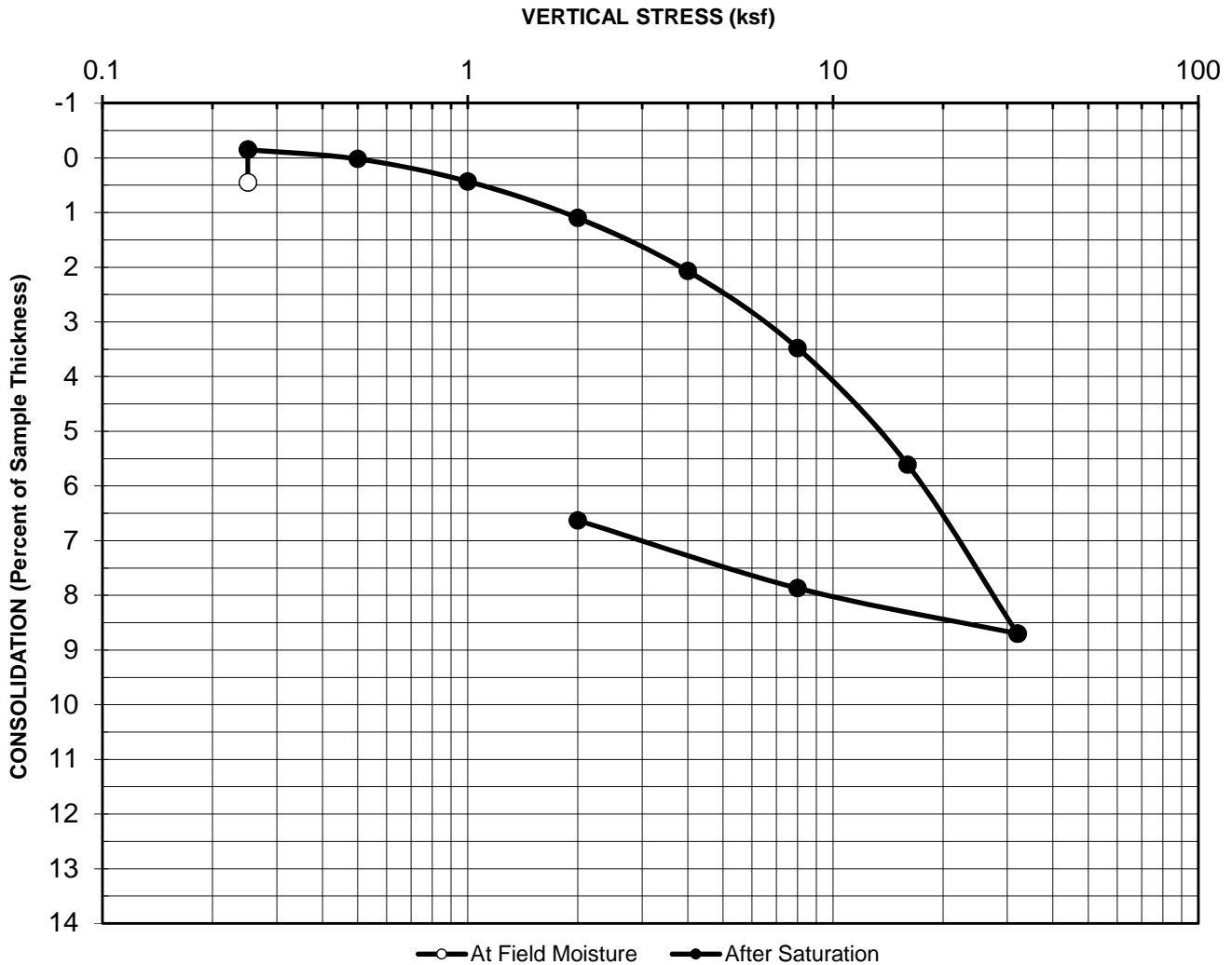


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Boring No. :	<u>GP-3</u>	Initial Dry Unit Weight (pcf):	<u>101.8</u>
Sample No.:	<u>14</u>	Initial Moisture Content (%):	<u>24.2</u>
Depth (feet):	<u>55.5</u>	Final Moisture Content (%):	<u>24.3</u>
Sample Type:	<u>Shelby Tube</u>	Assumed Specific Gravity:	<u>2.7</u>
Soil Description:	<u>Sandy Lean Clay w/some caliche</u>	Initial Void Ratio:	<u>0.66</u>
Remarks:	<u>Swell= 0.60% upon inundation</u>		

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name:	<u>1056 La Cienega Blvd</u>
Project No.:	<u>21086A</u>
Date:	<u>11/8/2021</u>
AP No:	<u>21-0873</u>
Sheet No:	<u>1</u>

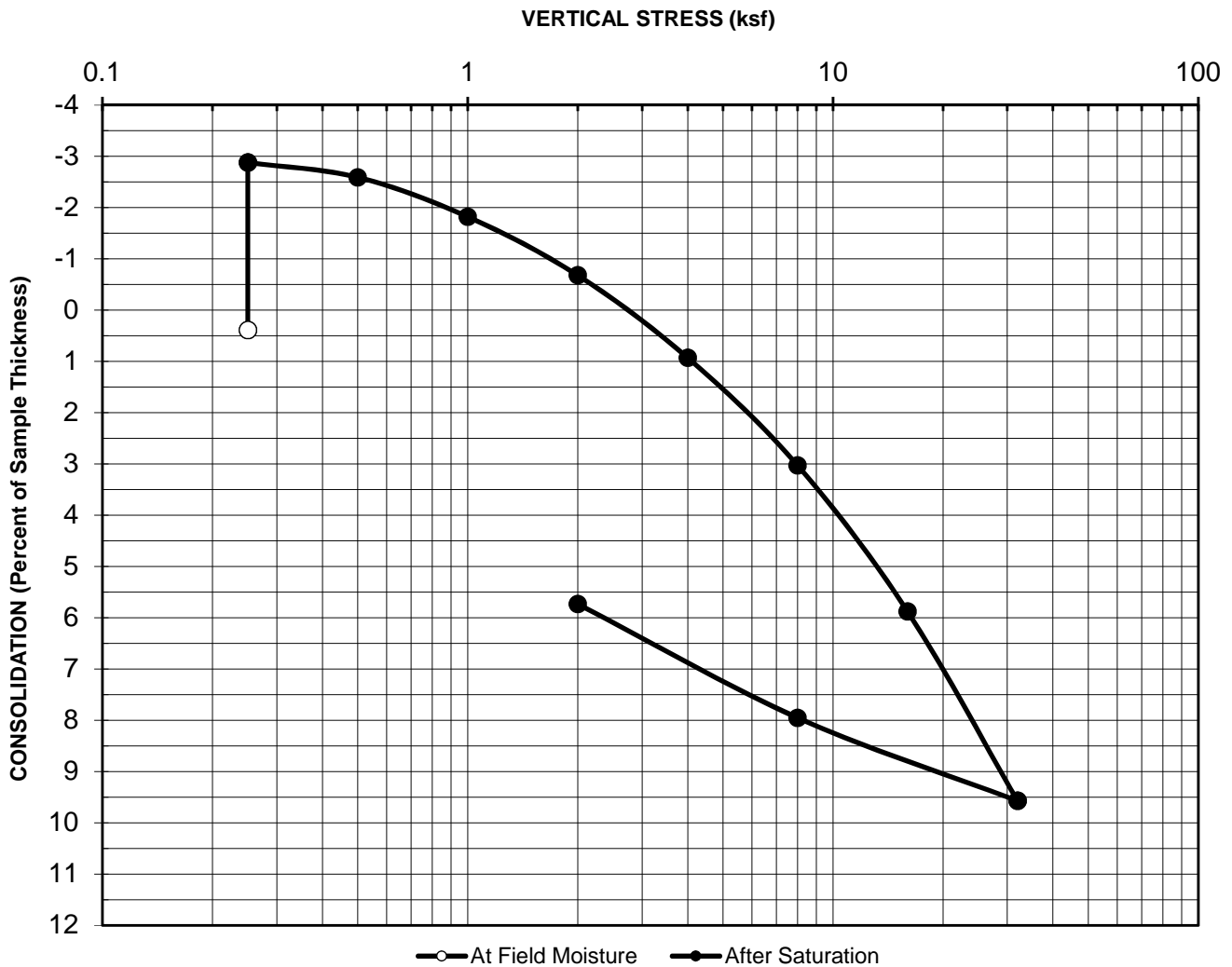


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Boring No. : GP-3

Initial Dry Unit Weight (pcf): 92.9

Sample No.: 23-top

Initial Moisture Content (%): 28.6

Depth (feet): 104.5

Final Moisture Content (%): 30.4

Sample Type: Shelby Tube

Assumed Specific Gravity: 2.7

Soil Description: Fat Clay

Initial Void Ratio: 0.81

Remarks: Swell= 3.27% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 11/8/2021

AP No: 21-0873 Sheet No: 1

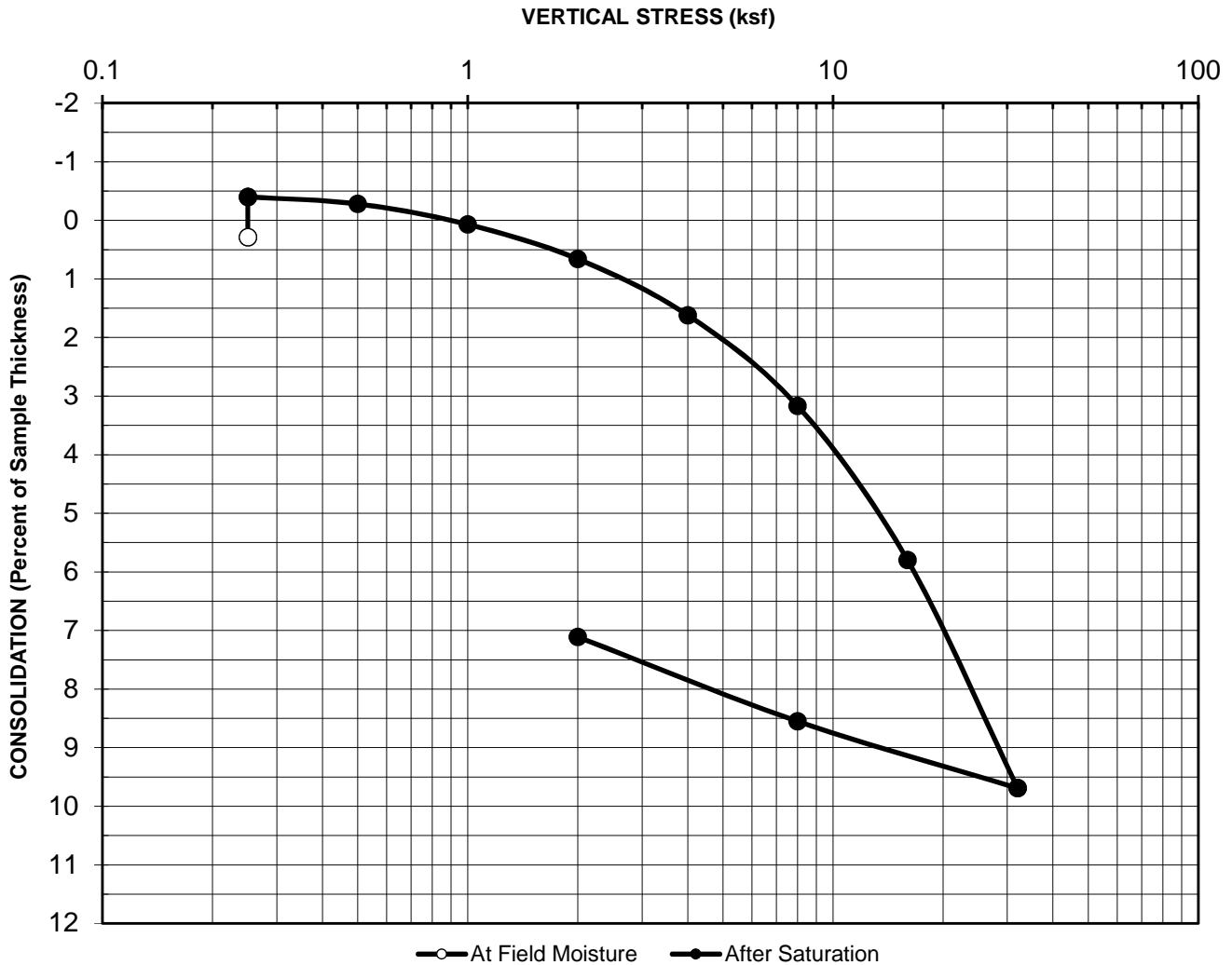


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Boring No. : GP-3

Initial Dry Unit Weight (pcf): 95.6

Sample No.: 28a

Initial Moisture Content (%): 28.2

Depth (feet): 139

Final Moisture Content (%): 28.3

Sample Type: Mod Cal

Assumed Specific Gravity: 2.7

Soil Description: Sandy Fat Clay

Initial Void Ratio: 0.76

Remarks: Swell= 0.69% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 11/8/2021

AP No: 21-0873 Sheet No: 1

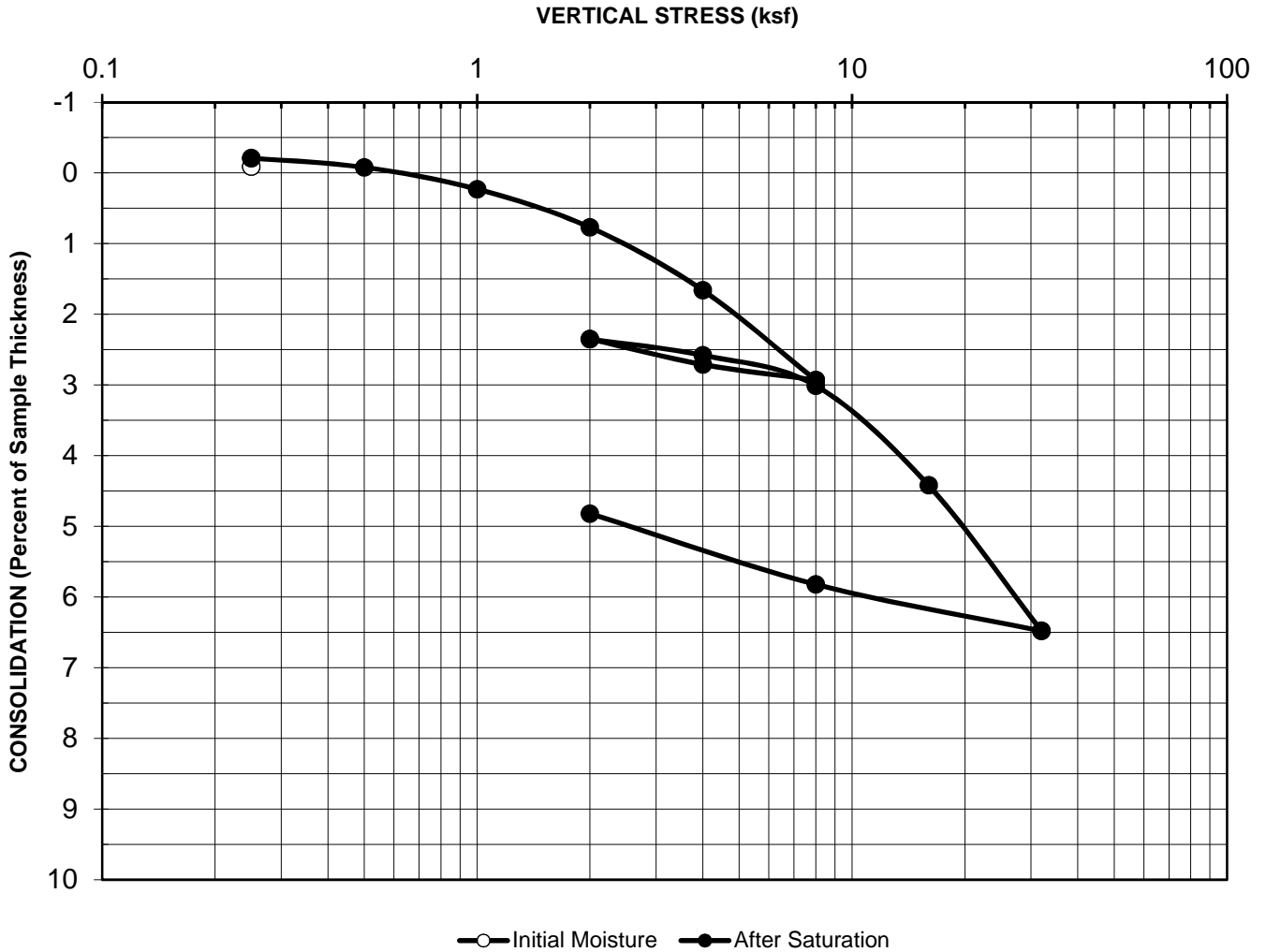


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Boring No. : GP-3

Initial Dry Unit Weight (pcf): 110.5

Sample No.: 11

Initial Moisture Content (%): 20.4

Depth (feet): 45.5

Final Moisture Content (%): 20.6

Sample Type: Shelby Tube

Assumed Specific Gravity: 2.9

Soil Description: Lean Clay w/sand

Initial Void Ratio: 0.64

Remarks: Swell= 0.12% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 11/08/21

AP No: 21-0873 **Sheet No:** 1

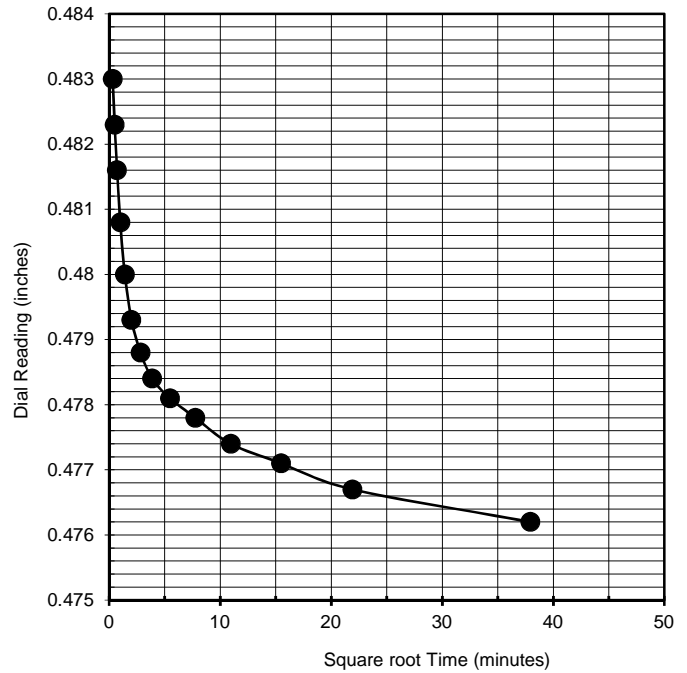
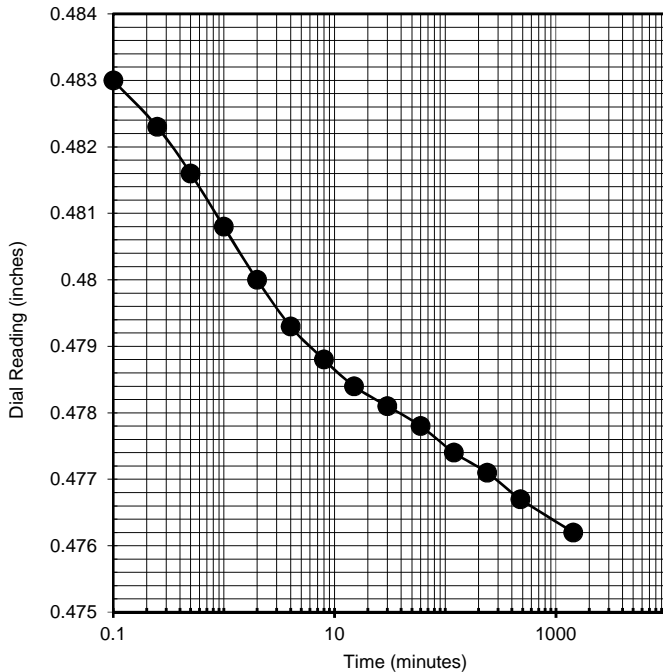


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Boring No. : GP-3
 Sample No.: 11
 Depth (feet): 45.5
 Loading Info: Initial Loading

Sample Type: Shelby Tube
 Soil Description: Lean Clay w/sand
 Vertical Pressure (ksf): 8
 Test Condition: Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.4830
0.25	0.4823
0.5	0.4816
1	0.4808
2	0.4800
4	0.4793
8	0.4788
15	0.4784
30	0.4781
60	0.4778
120	0.4774
240	0.4771
480	0.4767
1440	0.4762

SQRT Time (minutes)	Dial Reading (inches)
0.3162	0.4830
0.5000	0.4823
0.7071	0.4816
1.0000	0.4808
1.4142	0.4800
2.0000	0.4793
2.8284	0.4788
3.8730	0.4784
5.4772	0.4781
7.7460	0.4778
10.9545	0.4774
15.4919	0.4771
21.9089	0.4767
37.9473	0.4762

**TIME RATE CONSOLIDATION CURVE
 ASTM D 2435**

Project Name: 1056 La Cienega Blvd
 Project No.: 21086A
 Date: 11/8/2021
 AP No: 21-0873 Sheet No: 2

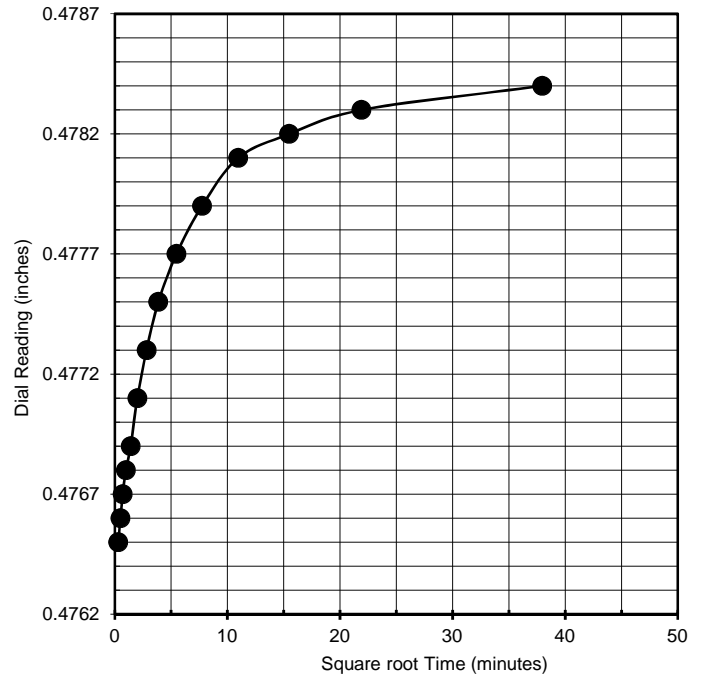
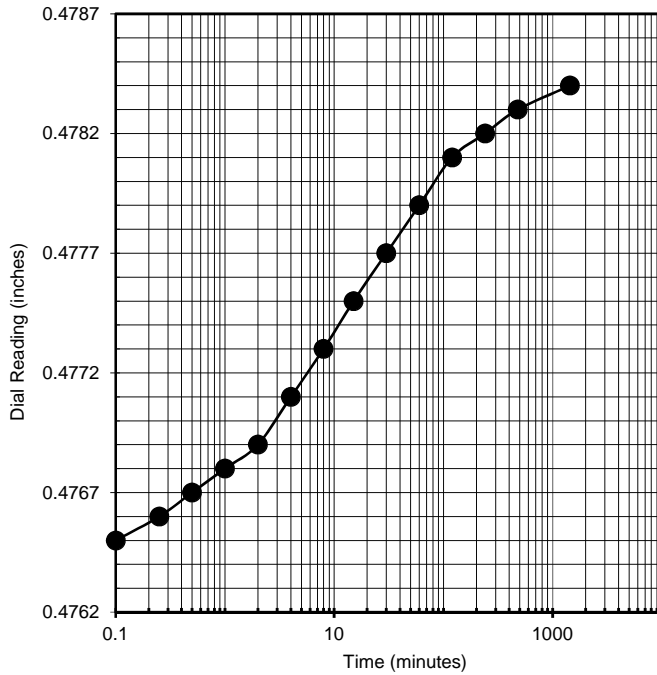


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Boring No. :	GP-3	Sample Type:	Shelby Tube
Sample No.:	11	Soil Description:	Lean Clay w/sand
Depth (feet):	45.5	Vertical Pressure (ksf):	4
Loading Info:	Initial Unloading	Test Condition:	Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.4765
0.25	0.4766
0.5	0.4767
1	0.4768
2	0.4769
4	0.4771
8	0.4773
15	0.4775
30	0.4777
60	0.4779
120	0.4781
240	0.4782
480	0.4783
1440	0.4784

SQRT Time (minutes)	Dial Reading (inches)
0.3162	0.4765
0.5000	0.4766
0.7071	0.4767
1.0000	0.4768
1.4142	0.4769
2.0000	0.4771
2.8284	0.4773
3.8730	0.4775
5.4772	0.4777
7.7460	0.4779
10.9545	0.4781
15.4919	0.4782
21.9089	0.4783
37.9473	0.4784

**TIME RATE CONSOLIDATION CURVE
ASTM D 2435**

Project Name:	1056 La Cienega Blvd
Project No.:	21086A
Date:	11/8/2021
AP No:	21-0873
Sheet No:	3

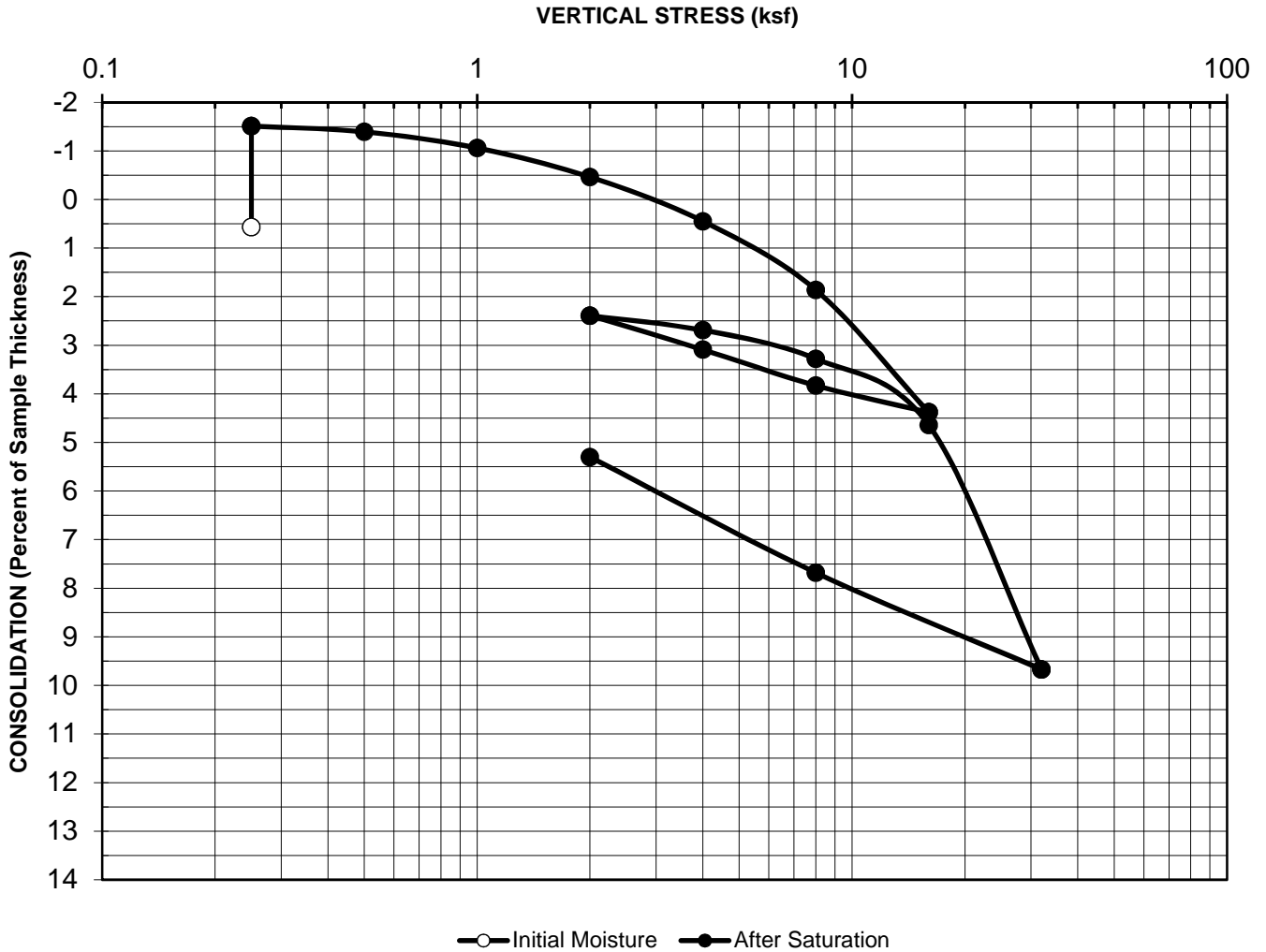


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Boring No. : GP-3

Initial Dry Unit Weight (pcf): 90.0

Sample No.: 22a

Initial Moisture Content (%): 32.3

Depth (feet): 99

Final Moisture Content (%): 33.9

Sample Type: Mod Cal

Assumed Specific Gravity: 2.9

Soil Description: Fat Clay

Initial Void Ratio: 1.01

Remarks: Swell= 2.08% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 11/08/21

AP No: 21-0873 **Sheet No:** 1

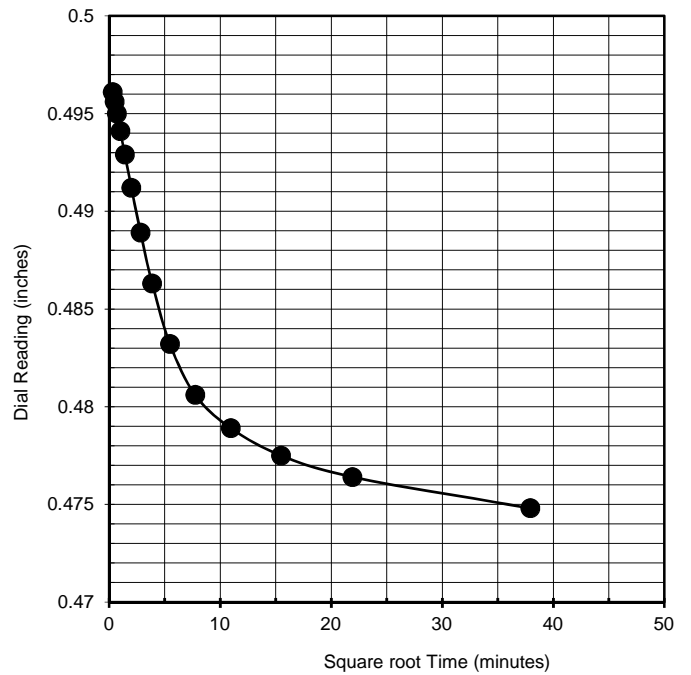
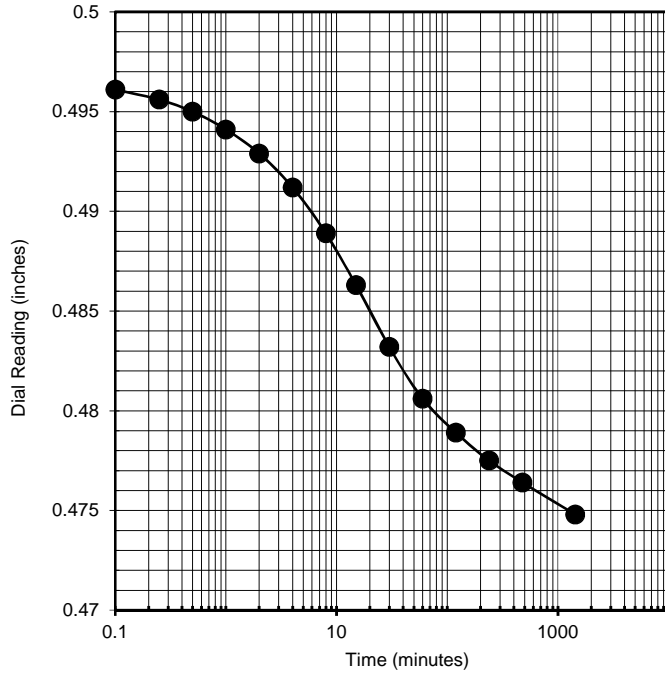


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Boring No. : GP-3
 Sample No.: 22a
 Depth (feet): 99
 Loading Info: Initial Loading

Sample Type: Mod Cal
 Soil Description: Fat Clay
 Vertical Pressure (ksf): 16
 Test Condition: Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.4961
0.25	0.4956
0.5	0.4950
1	0.4941
2	0.4929
4	0.4912
8	0.4889
15	0.4863
30	0.4832
60	0.4806
120	0.4789
240	0.4775
480	0.4764
1440	0.4748

SQRT Time (minutes)	Dial Reading (inches)
0.3162	0.4961
0.5000	0.4956
0.7071	0.4950
1.0000	0.4941
1.4142	0.4929
2.0000	0.4912
2.8284	0.4889
3.8730	0.4863
5.4772	0.4832
7.7460	0.4806
10.9545	0.4789
15.4919	0.4775
21.9089	0.4764
37.9473	0.4748

**TIME RATE CONSOLIDATION CURVE
 ASTM D 2435**

Project Name: 1056 La Cienega Blvd
 Project No.: 21086A
 Date: 11/8/2021
 AP No: 21-0873 Sheet No: 2

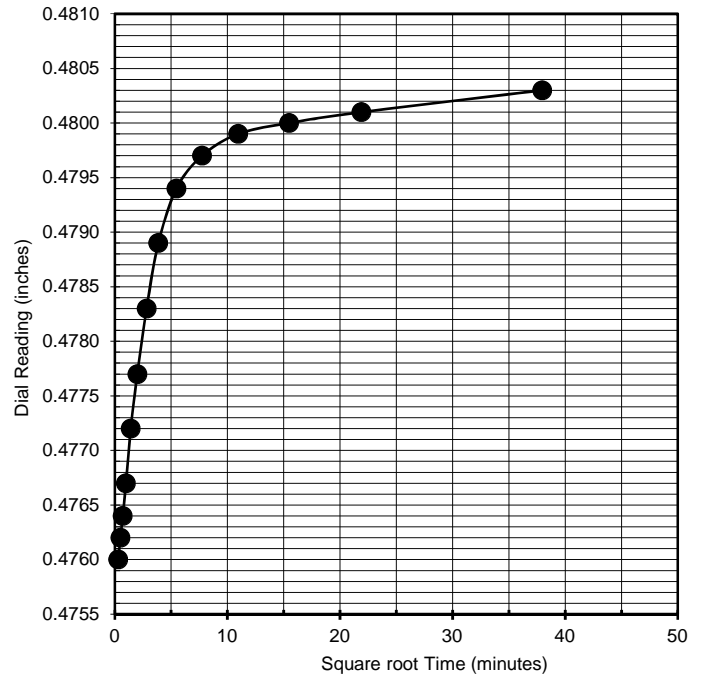
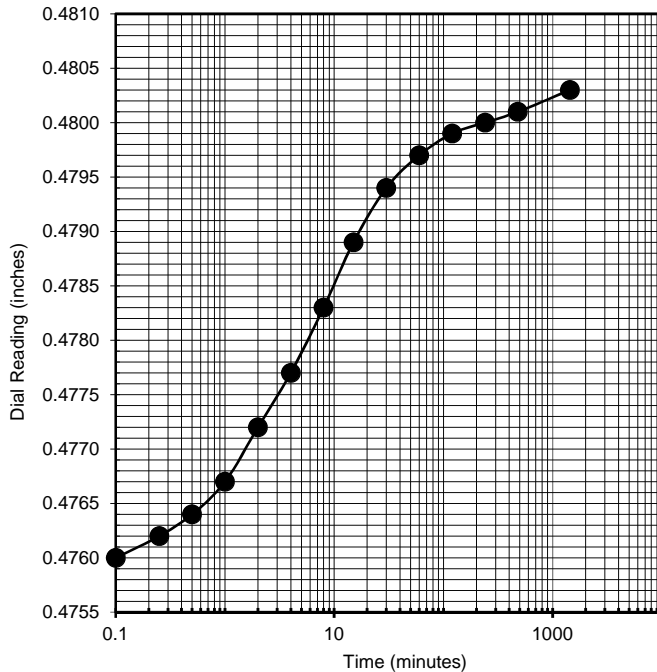


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Boring No. :	GP-3	Sample Type:	Mod Cal
Sample No.:	22a	Soil Description:	Fat Clay
Depth (feet):	99	Vertical Pressure (ksf):	8
Loading Info:	Initial Unloading	Test Condition:	Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.4760
0.25	0.4762
0.5	0.4764
1	0.4767
2	0.4772
4	0.4777
8	0.4783
15	0.4789
30	0.4794
60	0.4797
120	0.4799
240	0.4800
480	0.4801
1440	0.4803

SQRT Time (minutes)	Dial Reading (inches)
0.3162	0.4760
0.5000	0.4762
0.7071	0.4764
1.0000	0.4767
1.4142	0.4772
2.0000	0.4777
2.8284	0.4783
3.8730	0.4789
5.4772	0.4794
7.7460	0.4797
10.9545	0.4799
15.4919	0.4800
21.9089	0.4801
37.9473	0.4803

**TIME RATE CONSOLIDATION CURVE
ASTM D 2435**

Project Name:	1056 La Cienega Blvd
Project No.:	21086A
Date:	11/8/2021
AP No:	21-0873
Sheet No:	3

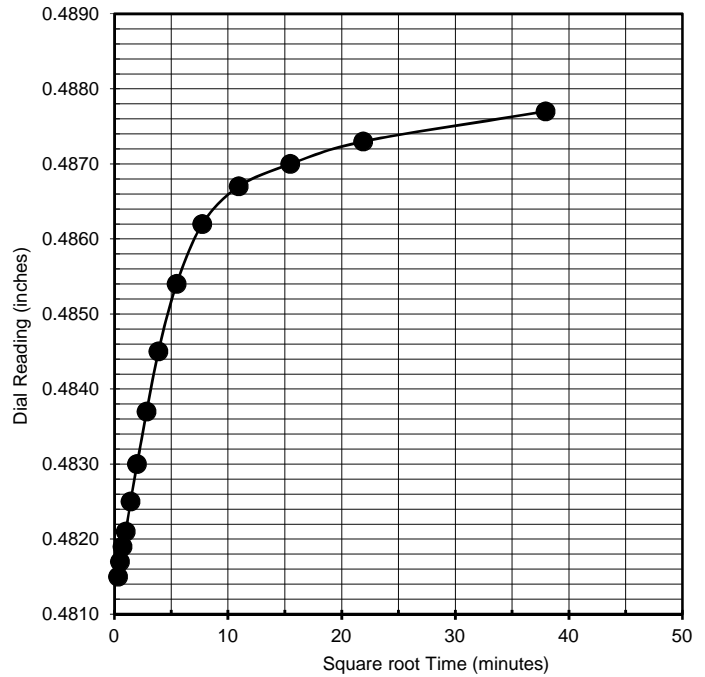
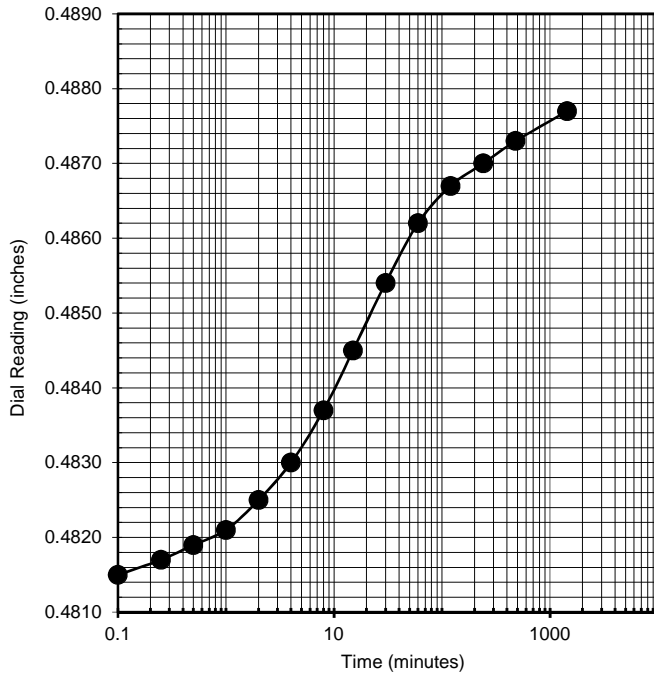


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Boring No. : GP-3
 Sample No.: 22a
 Depth (feet): 99
 Loading Info: Initial Unloading

Sample Type: Mod Cal
 Soil Description: Fat Clay
 Vertical Pressure (ksf): 4
 Test Condition: Saturated

Time (minutes)	Dial Reading (inches)
0.1	0.4815
0.25	0.4817
0.5	0.4819
1	0.4821
2	0.4825
4	0.4830
8	0.4837
15	0.4845
30	0.4854
60	0.4862
120	0.4867
240	0.4870
480	0.4873
1440	0.4877

SQRT Time (minutes)	Dial Reading (inches)
0.3162	0.4815
0.5000	0.4817
0.7071	0.4819
1.0000	0.4821
1.4142	0.4825
2.0000	0.4830
2.8284	0.4837
3.8730	0.4845
5.4772	0.4854
7.7460	0.4862
10.9545	0.4867
15.4919	0.4870
21.9089	0.4873
37.9473	0.4877

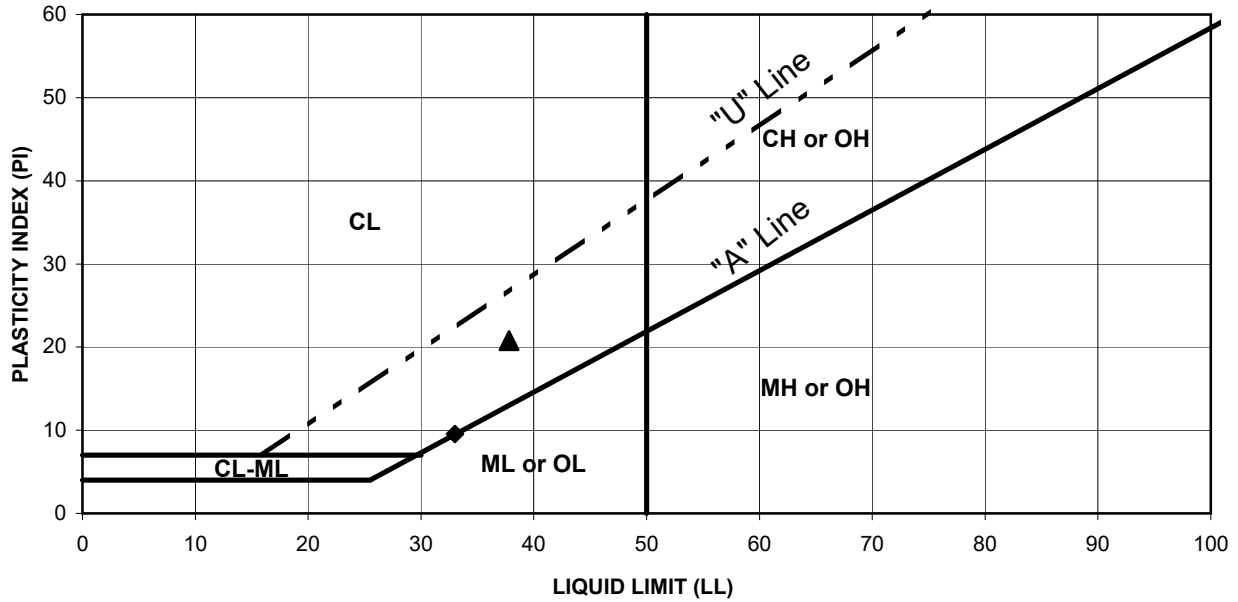
**TIME RATE CONSOLIDATION CURVE
 ASTM D 2435**

Project Name: 1056 La Cienega Blvd
 Project No.: 21086A
 Date: 11/8/2021
 AP No: 21-0873 Sheet No: 4



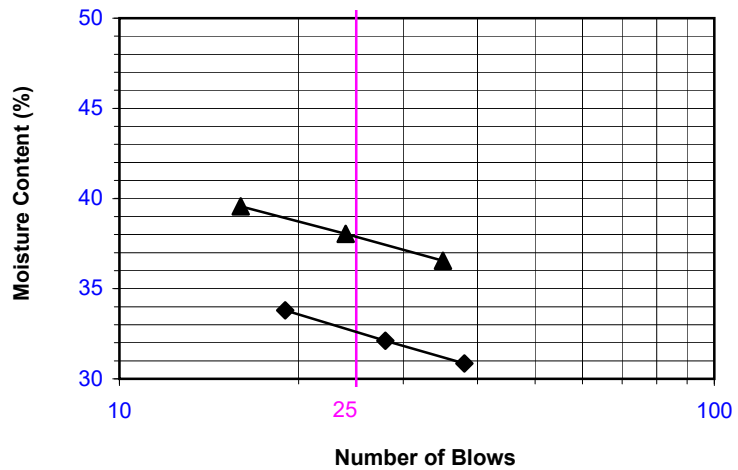
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>DK</u>	Date: <u>03/07/22</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>03/08/22</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>03/09/22</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test

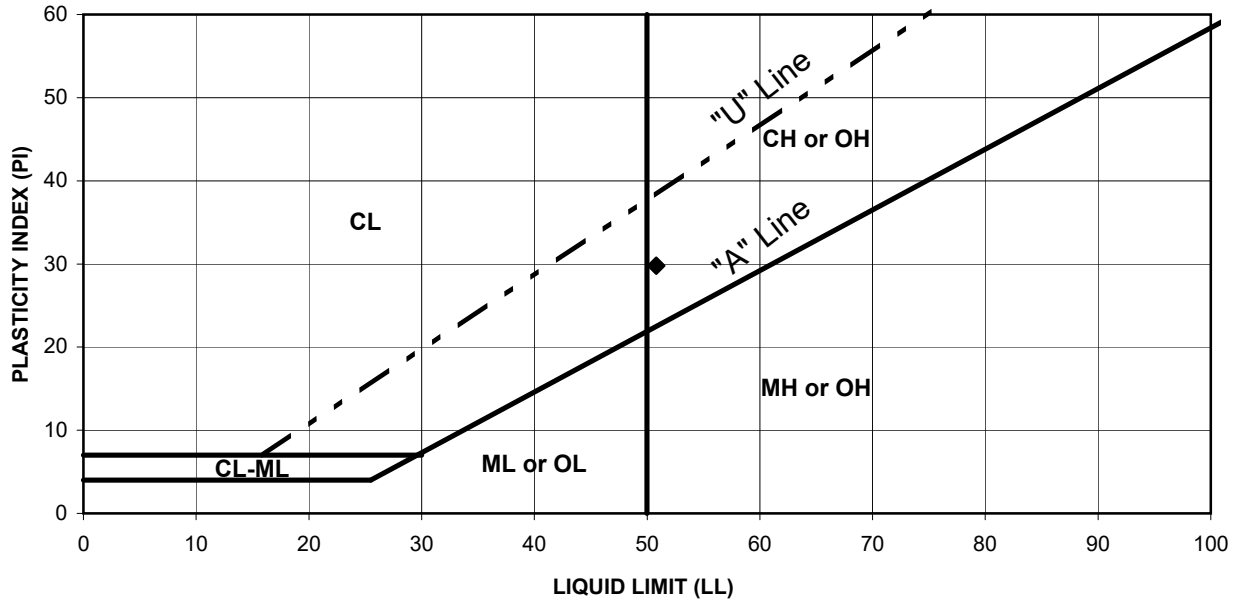


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-4	6	30	33	23	10	CL
▲	GP-4	9	45	38	17	21	CL



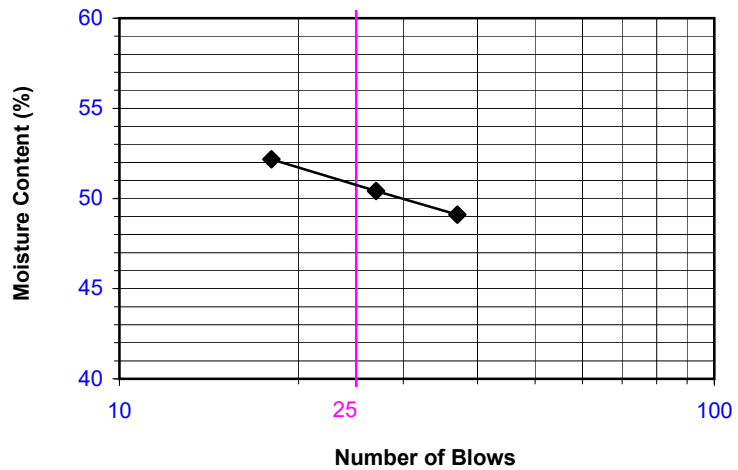
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>DK</u>	Date: <u>03/07/22</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>03/08/22</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>03/09/22</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test



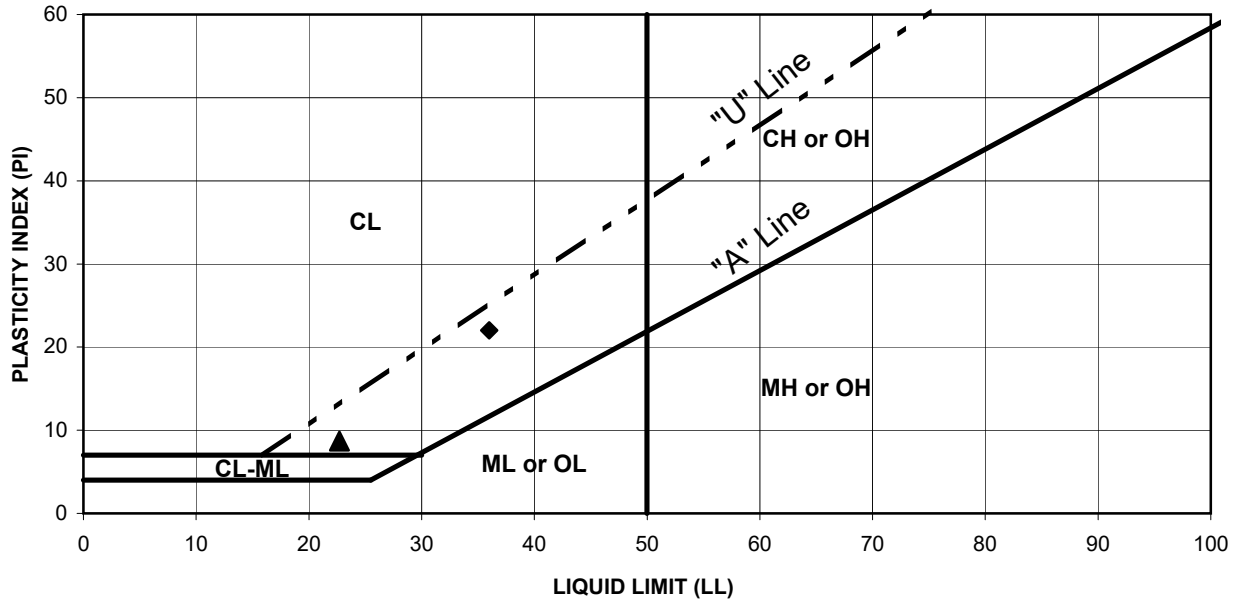
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-4	16	80	51	21	30	CH
	GP-4	20	100	NP	NP	NP	

* NP denotes "non-plastic"



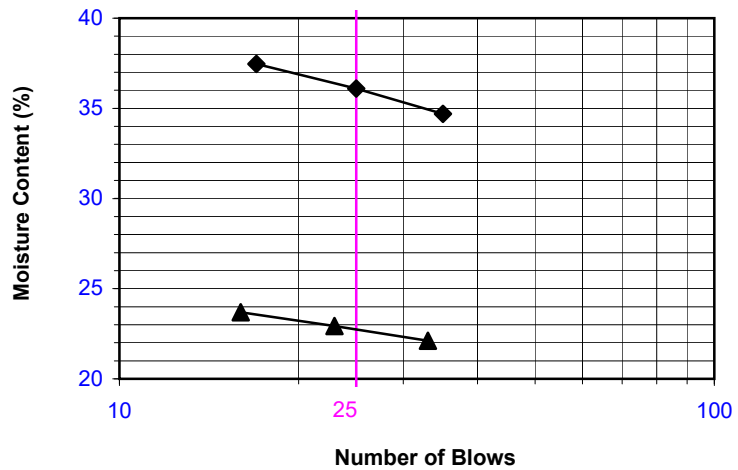
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>DK</u>	Date: <u>03/07/22</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>03/08/22</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>03/09/22</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test

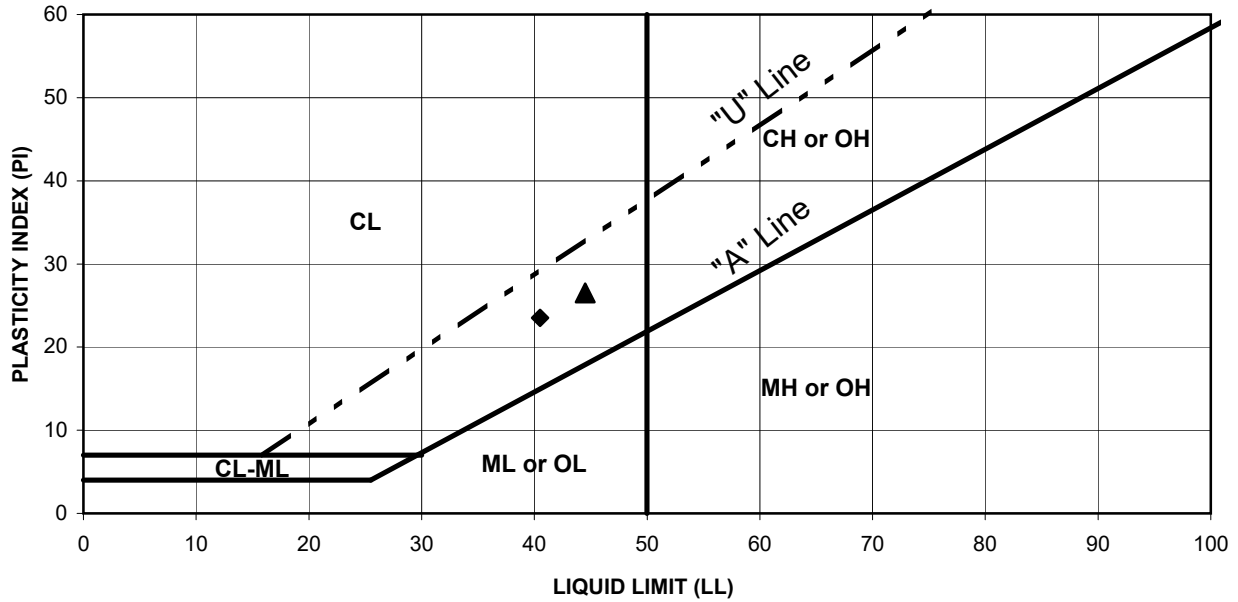


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-4	24	120	36	14	22	CL
▲	GP-4	26	130	23	14	9	CL



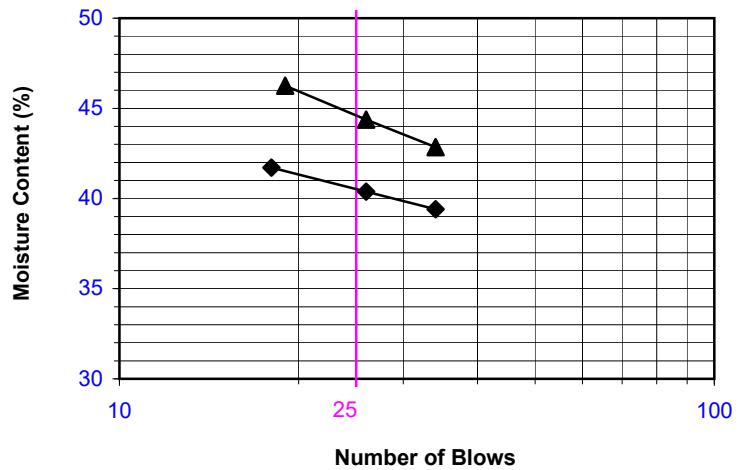
ATTERBERG LIMITS ASTM D 4318

Client Name: <u>GeoPentech</u>	Tested By: <u>DK</u>	Date: <u>03/07/22</u>
Project Name: <u>1056 La Cienega Blvd</u>	Computed By: <u>NR</u>	Date: <u>03/08/22</u>
Project No.: <u>21086A</u>	Checked By: <u>AP</u>	Date: <u>03/09/22</u>



PROCEDURE USED

- Wet Preparation
- Dry Preparation
- Procedure A
Multipoint Test
- Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	GP-4	28	140	41	17	24	CL
▲	GP-4	30	150	45	18	27	CL

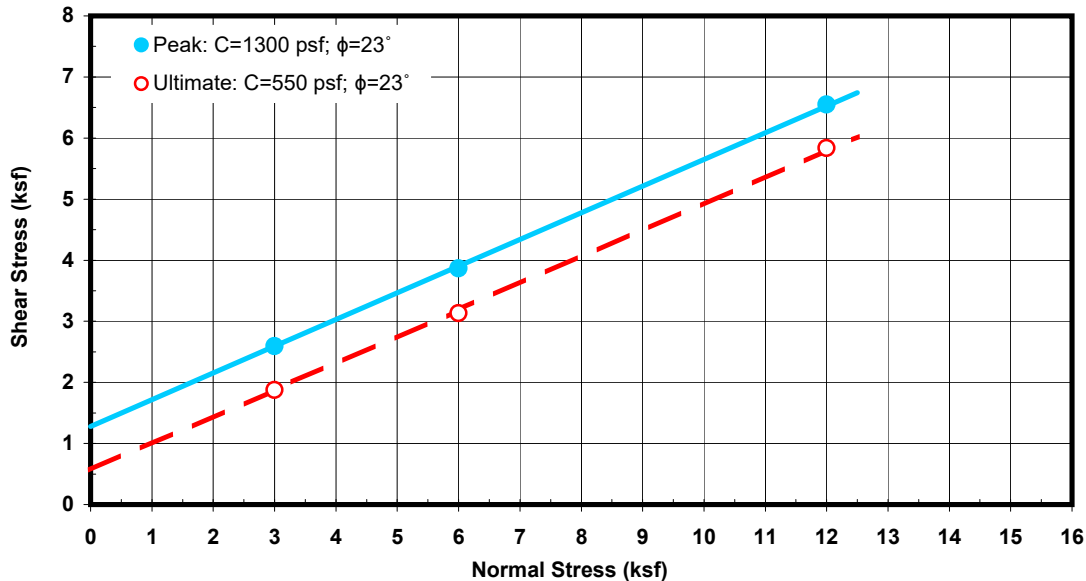
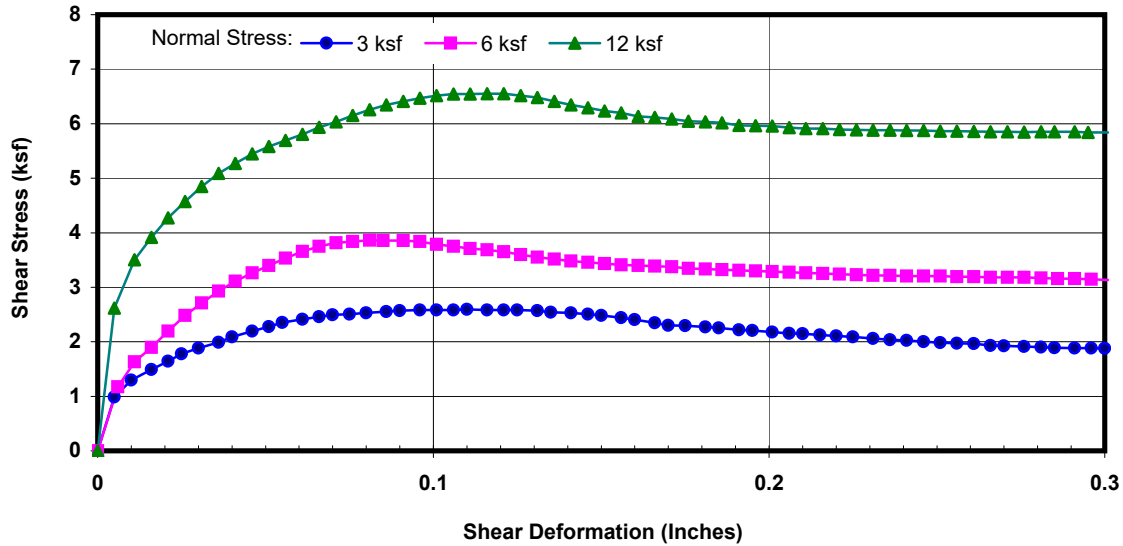


DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 1056 La Cienega Blvd
Project No.: 21086A
Boring No.: GP-4
Sample No.: 10 **Depth (ft):** 50
Sample Type: Mod. Cal.
Soil Description: Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 03/07/22
Computed By: NR **Date:** 03/09/22
Checked by: AP **Date:** 03/09/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
120.2	93.8	28.2	29.4	95	100	3	2.592	1.876
						6	3.864	3.132
						12	6.549	5.838





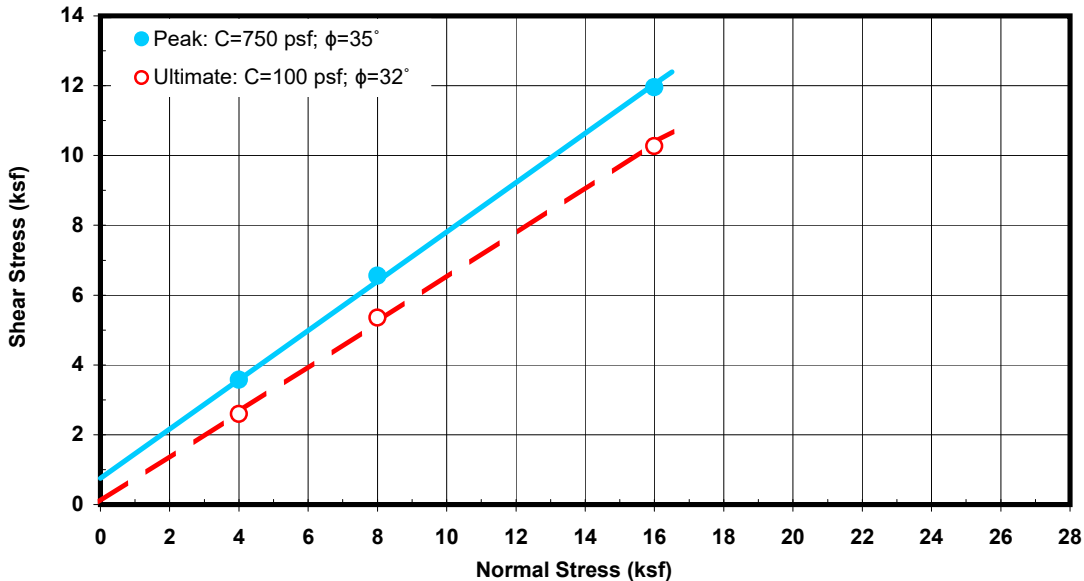
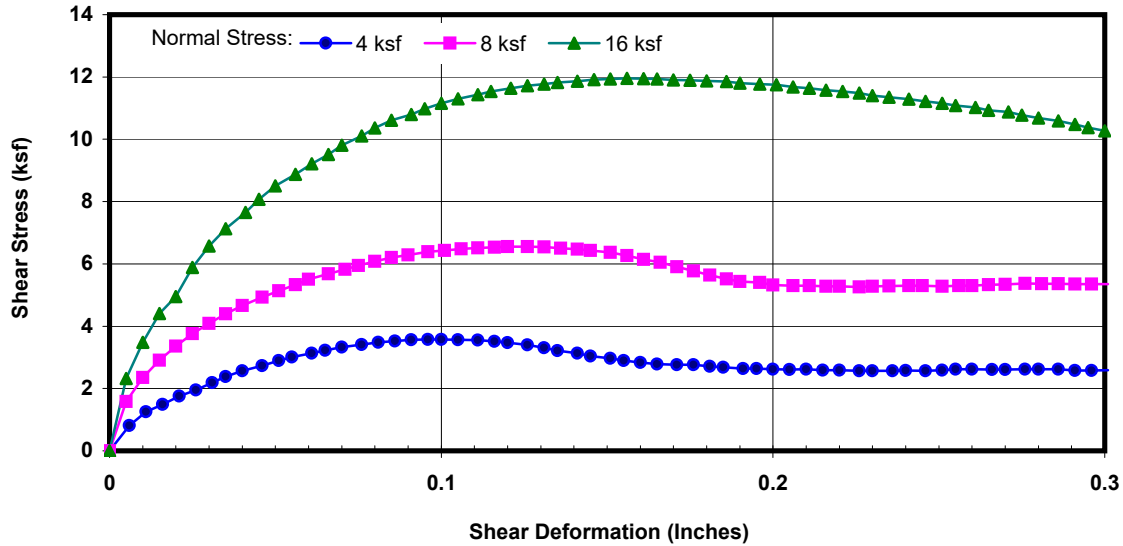
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DIRECT SHEAR TEST RESULTS ASTM D 3080

Client: GeoPentech
Project Name: 1056 La Cienega Blvd
Project No.: 21086A
Boring No.: GP-4
Sample No.: 14 **Depth (ft):** 70
Sample Type: Mod. Cal.
Soil Description: Sand w/silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 03/08/22
Computed By: NR **Date:** 03/09/22
Checked by: AP **Date:** 03/09/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
132.7	112.6	17.9	18.4	97	100	4	3.576	2.592
						8	6.552	5.352
						16	11.958	10.269



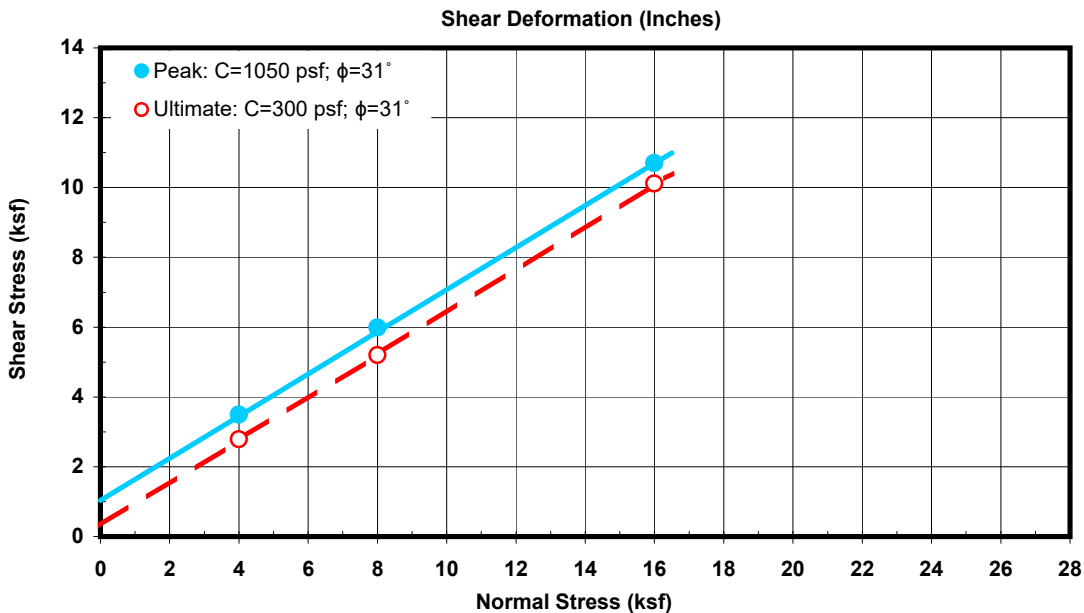
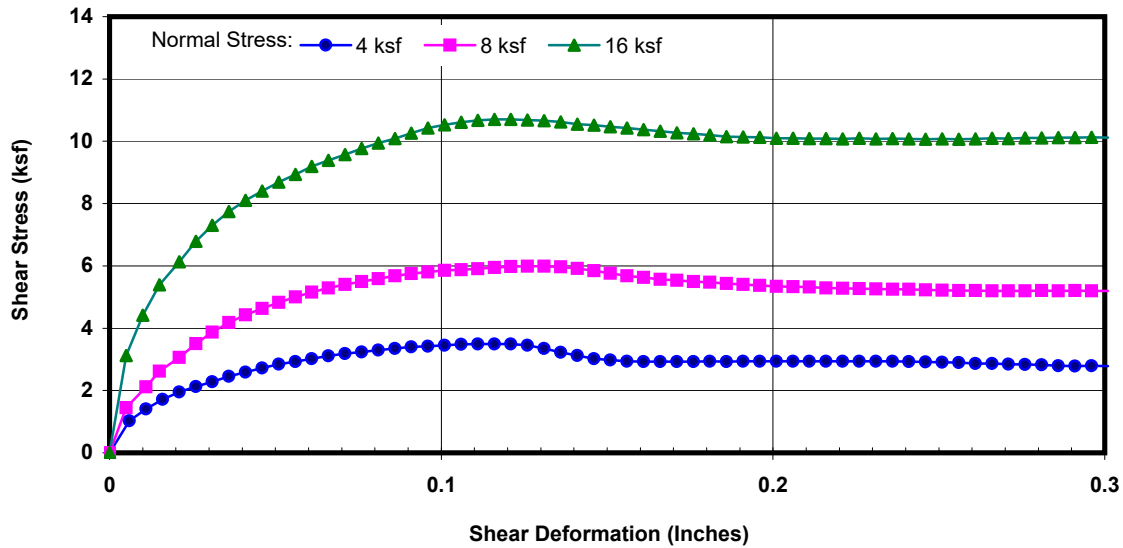


DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: GeoPentech
Project Name: 1056 La Cienega Blvd
Project No.: 21086A
Boring No.: GP-4
Sample No.: 22 **Depth (ft):** 110
Sample Type: Mod. Cal.
Soil Description: Sandy Silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 03/08/22
Computed By: NR **Date:** 03/09/22
Checked by: AP **Date:** 03/09/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
122.3	95.5	28.1	28.4	99	100	4	3.492	2.784
						8	5.988	5.198
						16	10.703	10.115



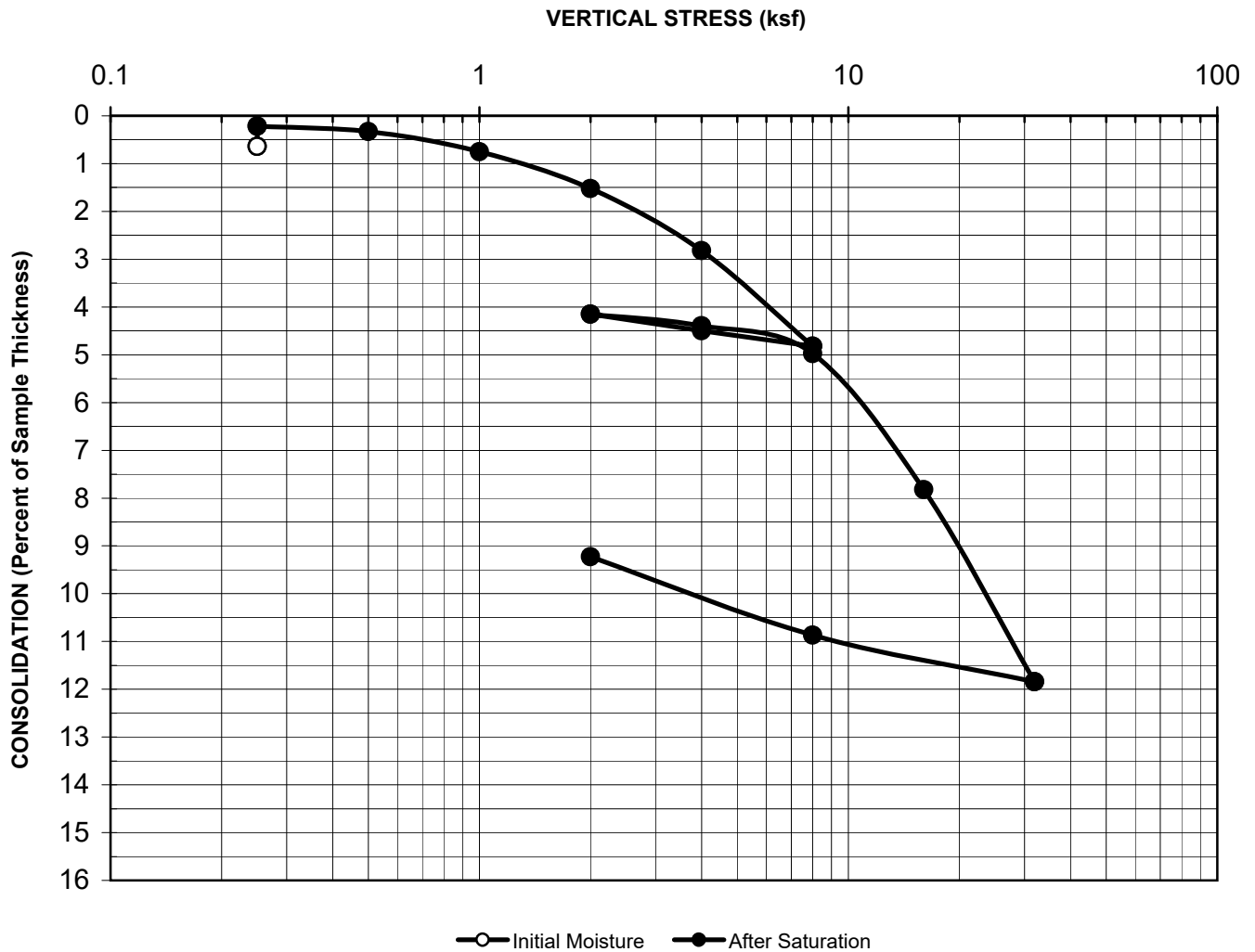


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Boring No. : GP-4

Initial Dry Unit Weight (pcf): 100.9

Sample No.: 9

Initial Moisture Content (%): 24.2

Depth (feet): 45

Final Moisture Content (%): 24.0

Sample Type: Mod Cal

Assumed Specific Gravity: 2.9

Soil Description: Lean Clay

Initial Void Ratio: 0.79

Remarks: Swell= 0.42% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 02/24/22

AP No: 21-0873 **Sheet No:** 1

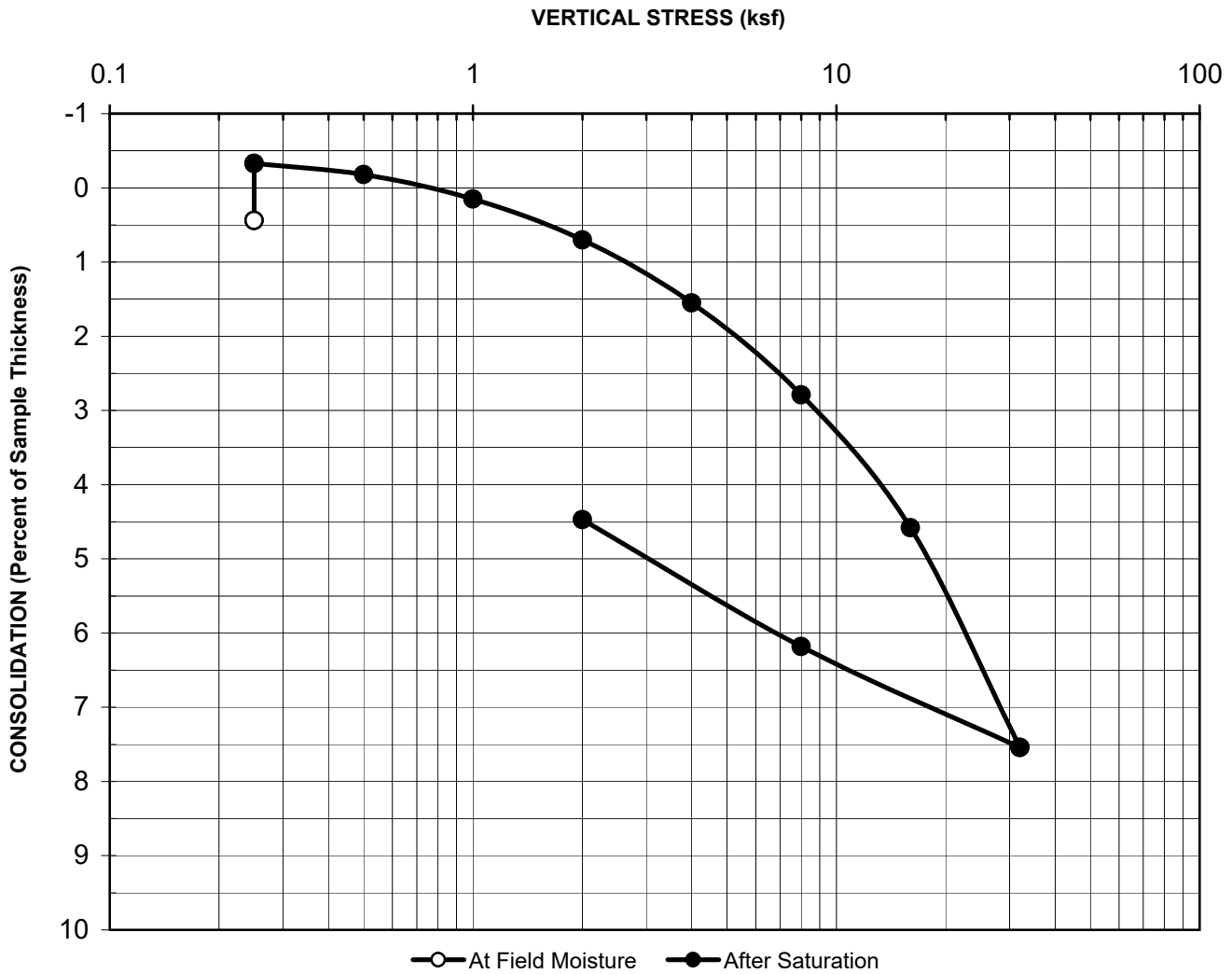


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Boring No. : GP-4

Initial Dry Unit Weight (pcf): 90.4

Sample No.: 16

Initial Moisture Content (%): 32.4

Depth (feet): 80

Final Moisture Content (%): 32.3

Sample Type: Mod Cal

Assumed Specific Gravity: 2.7

Soil Description: Fat Clay

Initial Void Ratio: 0.86

Remarks: Swell= 0.77% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 2/24/2022

AP No: 21-0873 Sheet No: 1

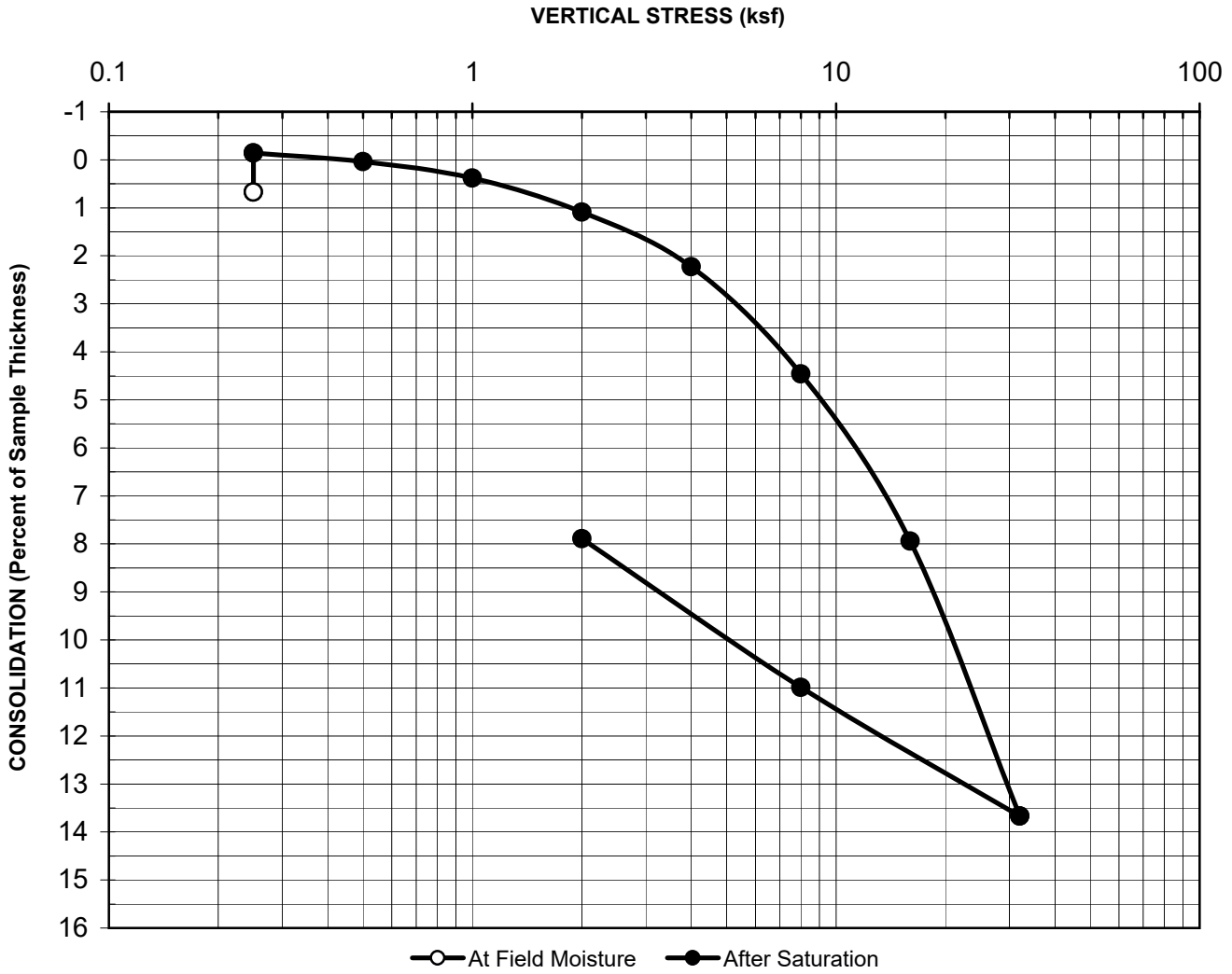


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Boring No. : GP-4

Initial Dry Unit Weight (pcf): 90.6

Sample No.: 31

Initial Moisture Content (%): 30.4

Depth (feet): 155

Final Moisture Content (%): 33.3

Sample Type: Mod Cal

Assumed Specific Gravity: 2.7

Soil Description: Sandy Fat Clay

Initial Void Ratio: 0.86

Remarks: Swell= 0.81% upon inundation

**CONSOLIDATION CURVE
ASTM D 2435**

Project Name: 1056 La Cienega Blvd

Project No.: 21086A

Date: 2/24/2022

AP No: 21-0873 Sheet No: 1

APPENDIX E

SURFACE WAVE GEOPHYSICAL MEASUREMENTS

E.1 INTRODUCTION

This appendix presents the results of the surface wave geophysical investigation performed in support of ground motion development and geotechnical design investigation for a proposed new building that includes a 27-story tower with one subterranean floor at 1022 to 1056 La Cienega Boulevard in Los Angeles, California. The geophysical investigation consisted of surface wave surveys using Multichannel Analysis of Surface Waves (MASW) and Refraction Microtremor (ReMi) methods. The geophysical measurements were performed along three survey lines (SW21-1 through SW21-3) at the locations shown in Figure E-1. The purpose of the geophysical surveys was to measure seismic shear-wave (S-wave) velocities at a range of depths to evaluate foundation properties (i.e. VS30) at the site. The geophysical data were collected and processed by a senior staff scientist under the supervision of a California-licensed Professional Geophysicist.

E.2 SURFACE WAVE GEOPHYSICAL METHODS

Both active and passive surface wave surveys were performed at the site. The active surface wave surveys were performed using MASW methods, and the passive surveys were performed using ReMi methods. A detailed description of MASW is provided in Park et al. (1999), and ReMi is described in Louie (2001) and Louie et al. (2021).

In general, the surface wave method records Rayleigh waves generated either with (1) an active source (e.g. sledgehammer) for the MASW method or (2) a passive (ambient) source (e.g. vehicular traffic) for the ReMi method. In a layered medium, Rayleigh surface waves of different frequencies (or wavelengths) propagate at different velocities, referred to as phase velocity. This phase velocity primarily depends on the material stiffness properties (e.g. S-wave velocity) over a depth approximately equal to one wavelength. Consequently, lower frequency, longer wavelength surface wave energy will provide samples to greater survey depths than higher frequency, shorter wavelength energy. Because surface waves of different frequencies (wavelengths) sample different depths, they travel at different velocities (dispersion) in a layered medium. Surface wave geophysical surveys measure the dispersive nature of the geologic medium and produce dispersion curves, which show the variation of Rayleigh wave phase velocity as a function of frequency (or wavelength). Due to the generally lower frequency nature of passive surface wave energy, passive surface wave techniques (i.e. ReMi) have the potential to supplement active surface wave data to achieve deeper investigation depths. For this reason, it is advantageous to perform both types of measurement along the same lines as was done for this project.

After the dispersion curve is generated, the dispersion curve picks are then iteratively fitted to a horizontally layered, laterally continuous, homogeneous-isotropic, S-wave velocity model that would account for the measured surface wave velocity dispersion. The results provide a representative average estimate of the one-dimensional S-wave velocity profile under the array (velocity vs. depth).

E.3 SURFACE WAVE GEOPHYSICAL PROCEDURES

The MASW and ReMi investigations were performed at the site on August 26, 2021. These measurements were collected using a Geometrics S12 seismograph with a linear array of twelve, 4.5-Hz geophones. As shown on Figure E-1, the three survey lines were performed across the currently existing empty lot at the site and along the sidewalk immediately west of the lot. Geophones were spaced at 10- and 20-foot intervals (110- and 220-foot line lengths) for line SW21-1 and SW21-3 MASW measurements and at only 10-foot intervals (110-foot line length) for line SW21-2 MASW measurements. ReMi measurements for SW21-1 and SW21-3 were collected from the same 20-foot-spaced geophone arrays as the MASW measurements, while ReMi measurements for SW21-2 were collected from the 10-foot-spaced array used for MASW.

For the MASW measurements, the active seismic source consisted of a sledgehammer blow to a ground plate. Shots were performed at station intervals equal to the geophone spacing (either 10 or 20 feet long), starting at the first geophone and finishing 3 to 5 station intervals (50 to 60 feet) behind the first geophone. At each shot location, the sledgehammer was hit three times, and the resultant waveforms were stacked. A 1,024-millisecond-long record (0.5 millisecond sample interval) was recorded at each shot location. The recorded MASW data were subsequently processed using the program SurfSeis by Kansas Geological Survey. This program performs a wavefield transformation to convert the seismic data from time-distance space to frequency-phase velocity space. The highest amplitude energy in the frequency-phase velocity space was selected for the dispersion curve.

Because of the typical lower frequency nature of passive surface wave energy, ReMi measurements were performed to supplement the MASW measurements to deeper investigation depths. A total of ten 32,768-millisecond-long ReMi records (2 millisecond sample interval) were collected at each survey location along the 220-foot 12-channel geophone array for SW21-1 and SW21-3 and the 110-foot array for line SW21-2. The source of ambient surface wave energy was primarily vehicular traffic in the area. The recorded ReMi data were processed using the program SeisOpt ReMi by Optim Software. This program performs a slowness-frequency waveform transformation to the recorded surface wave data to separate Rayleigh waves from other seismic arrivals. The ReMi dispersion curves are picked as the lower bound envelope of the surface wave energy, which represents the slowest surface wave energy (highest slowness). In theory, the slowest identifiable surface wave energy represents the energy that is propagating parallel to the survey line. Energy propagating oblique to the line would be observed as having a higher velocity.

For each line, the ReMi dispersion curve picks were combined with the dispersion curve picks generated from MASW for modeling. The degree of fit of the overlapping ReMi and MASW dispersion picks provided confidence in the results. Additionally, as noted above, the ReMi and MASW data complement each other by generally sampling different frequency ranges of surface wave data. After the data were combined, a best fit polynomial dispersion curve was calculated for modeling. The best fit dispersion curve was then iteratively fitted to a one-dimensional S-wave velocity model using the SurfSeis software. The results provide a one-dimensional vertical profile of S-wave velocity as a function of depth averaged beneath the extent of the survey line.

E.4 SURFACE WAVE GEOPHYSICAL RESULTS

The results of the combined MASW and ReMi surface wave measurements are presented in Figures E-2 through E-4 for lines SW21-1 through SW21-3, respectively. These figures present the MASW, ReMi, and best fit surface wave dispersion curves and the corresponding representative S-wave velocity models. As seen in these figures, the MASW and ReMi dispersion curves are generally in good agreement in the regions that overlap.

Figure E-5 summarizes the surface wave measurement results for the site. This figure shows (1) the S wave velocity models for lines SW21-1 through SW21-3 plotted as a function of depth below ground surface and (2) the site average S-wave velocity for all the measurements calculated at 1-foot increments.

Based on the results shown on Figure E-5, the VS30 was calculated based on the procedures outlined in the National Earthquake Hazards Reduction Program (NEHRP) and UBC. The VS30 was calculated from the following equation from these references:

$$v_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}}$$

where:

i = distinct different soil and/or rock layer between 1 and n

v_{si} = shear wave velocity in feet per second of layer i

d_i = thickness of any layer within the 100-foot interval

$\sum_{i=1}^n d_i = 100$ feet

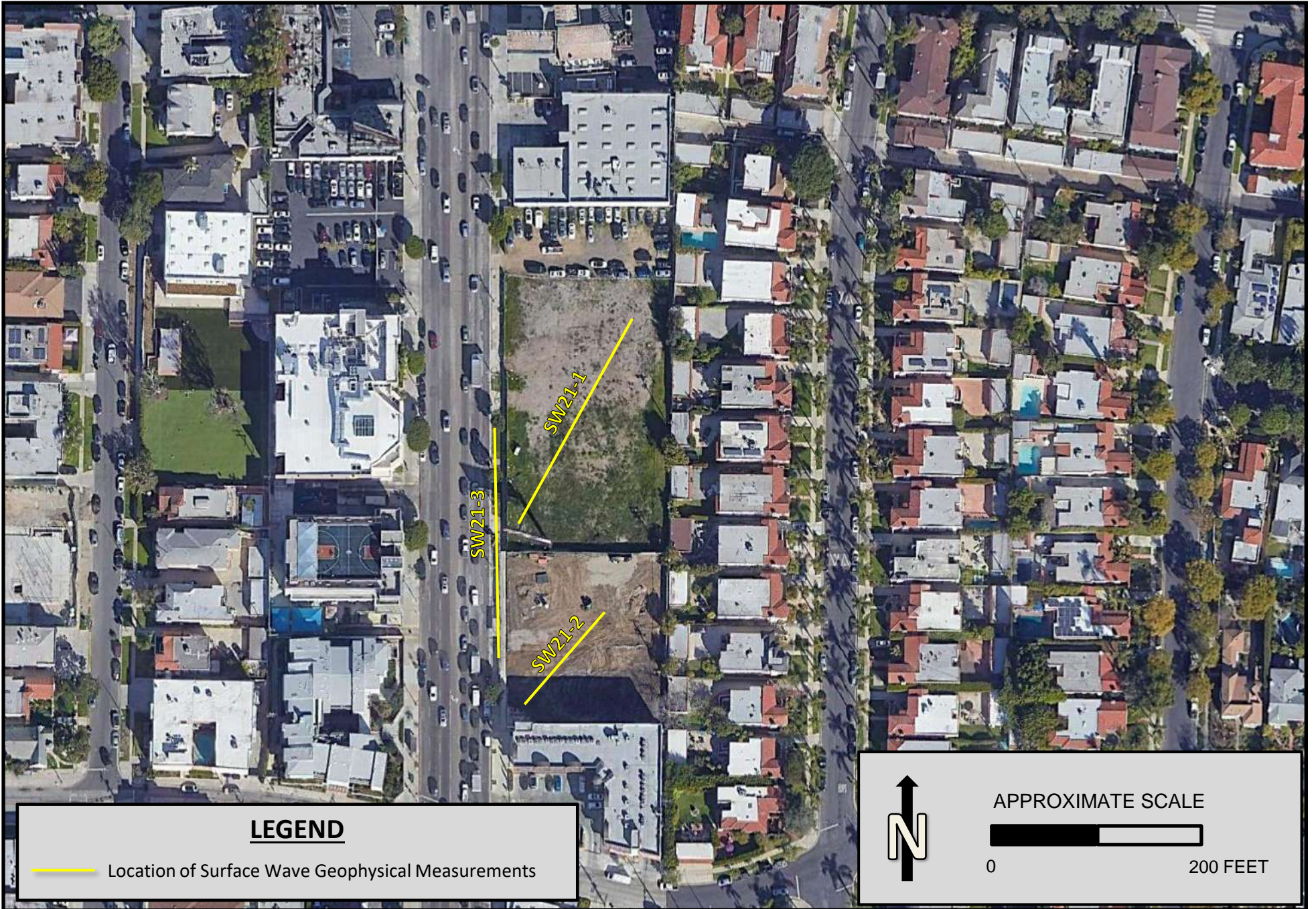
Based on this procedure, the site average VS30 was calculated from ground surface to 100 feet below ground surface. The VS30 below ground surface was calculated as 965 ft/s (294 m/s), which corresponds with NEHRP Site Class D, stiff soil ($600 < VS30 \leq 1,200$ ft/s). VS30 calculations for depth intervals starting below ground surface are also shown on Figure E-5.

E.5 REFERENCES

Louie, J.N. (2001). Faster, Better: Shear-wave Velocity to 100 Meters Depth from Refraction Microtremor Arrays: Bulletin of the Seismological Society of America, v. 91, no. 2, p. 347-364.

Louie, J.N., Panchar, A., and Kissane, B. (2021). Guidelines and pitfalls of refraction microtremor surveys: Journal of Seismology, published online June 7, 2021, <https://doi.org/10.1007/s10950-021-10020-5>.

Park, C.B., Miller, R.D., and Xia, J. (1999). Multichannel analysis of surface waves: Geophysics, v. 64, no. 3, pp. 800-808.

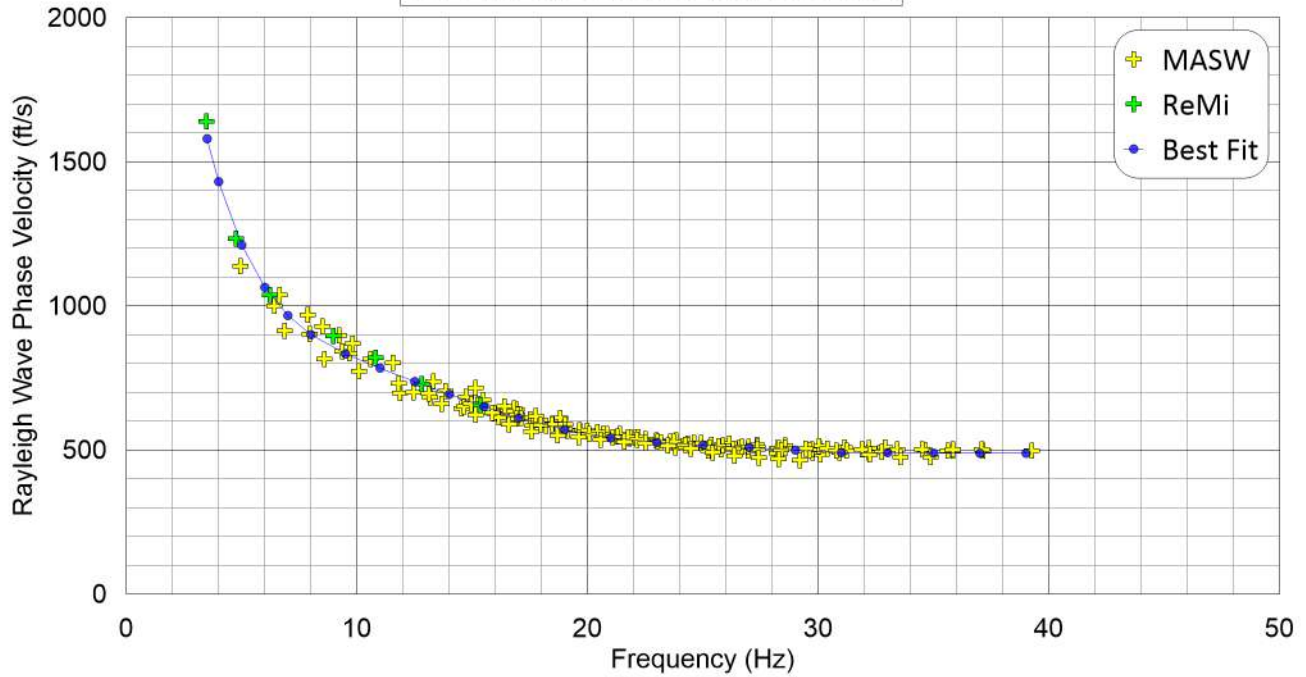


LEGEND

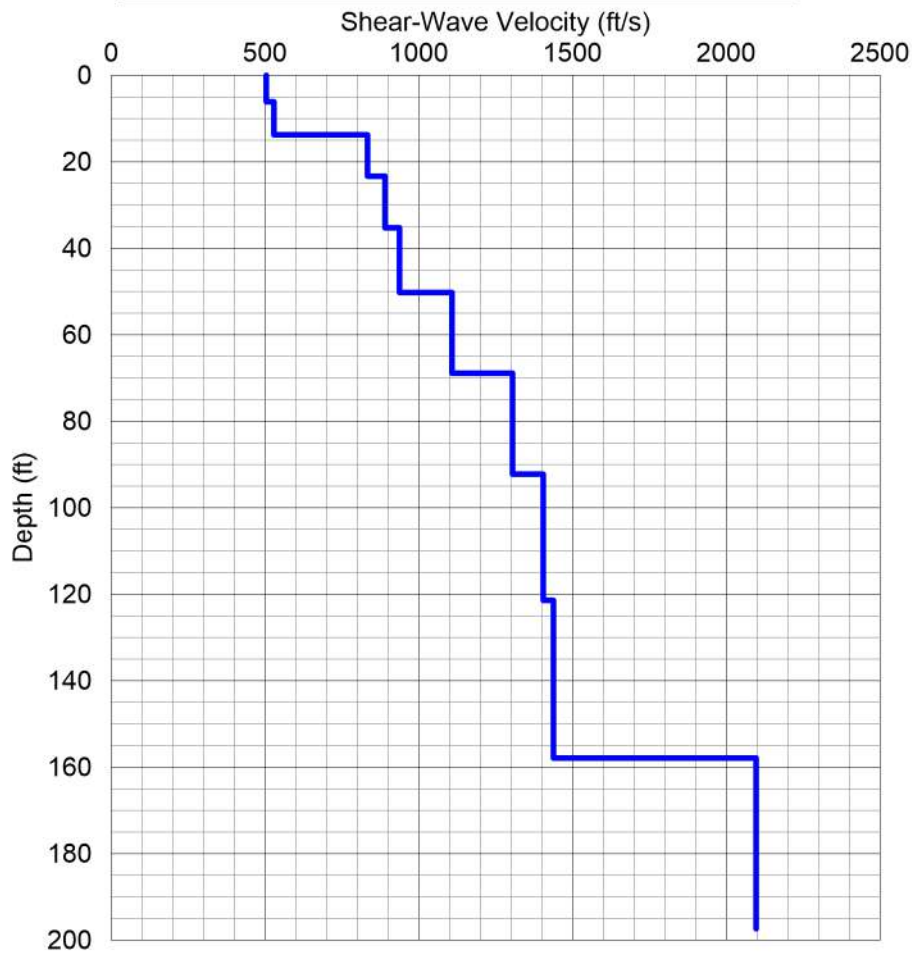
— Location of Surface Wave Geophysical Measurements

SITE MAP

SURFACE WAVE DISPERSION CURVE

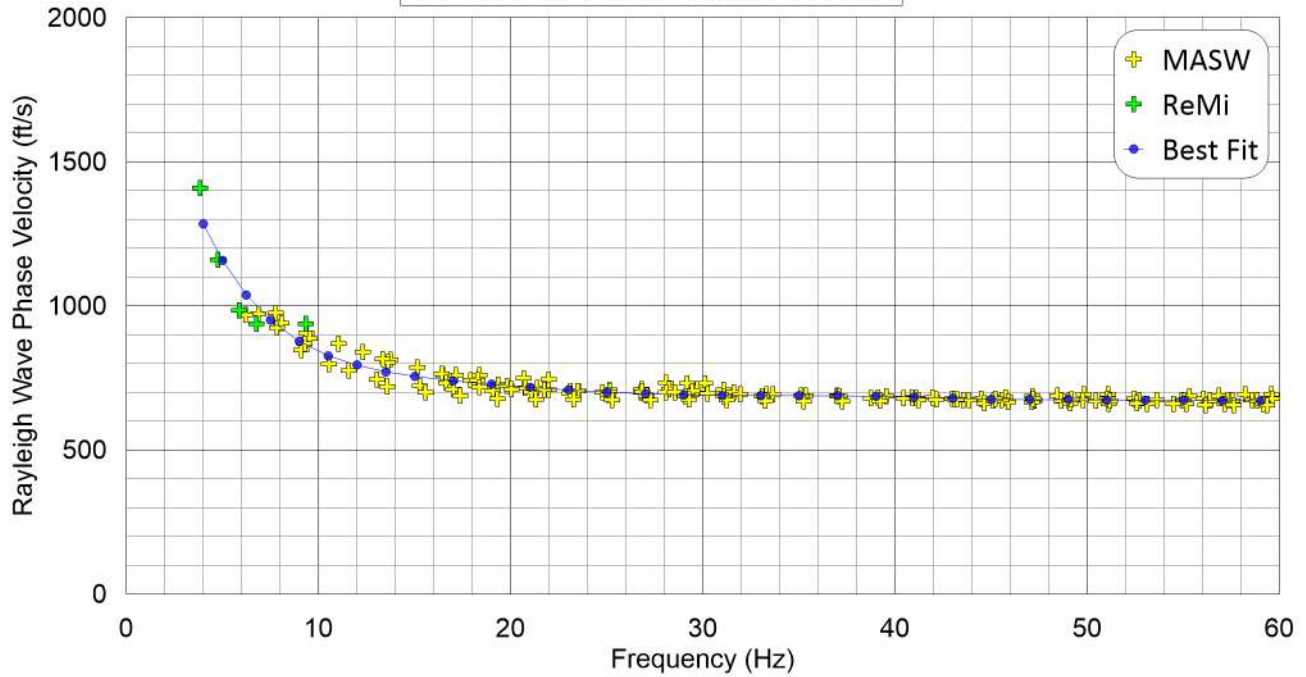


REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL

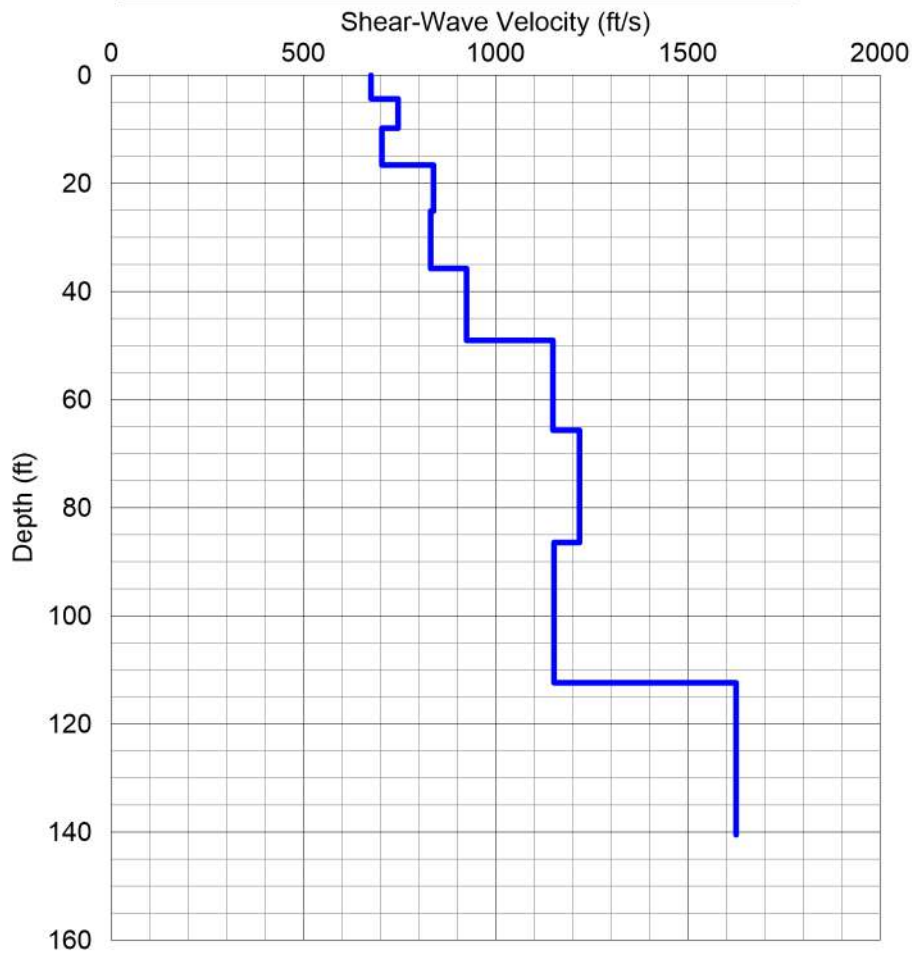


SW21-1: S-WAVE VELOCITY COMBINED SOURCE MODEL

SURFACE WAVE DISPERSION CURVE

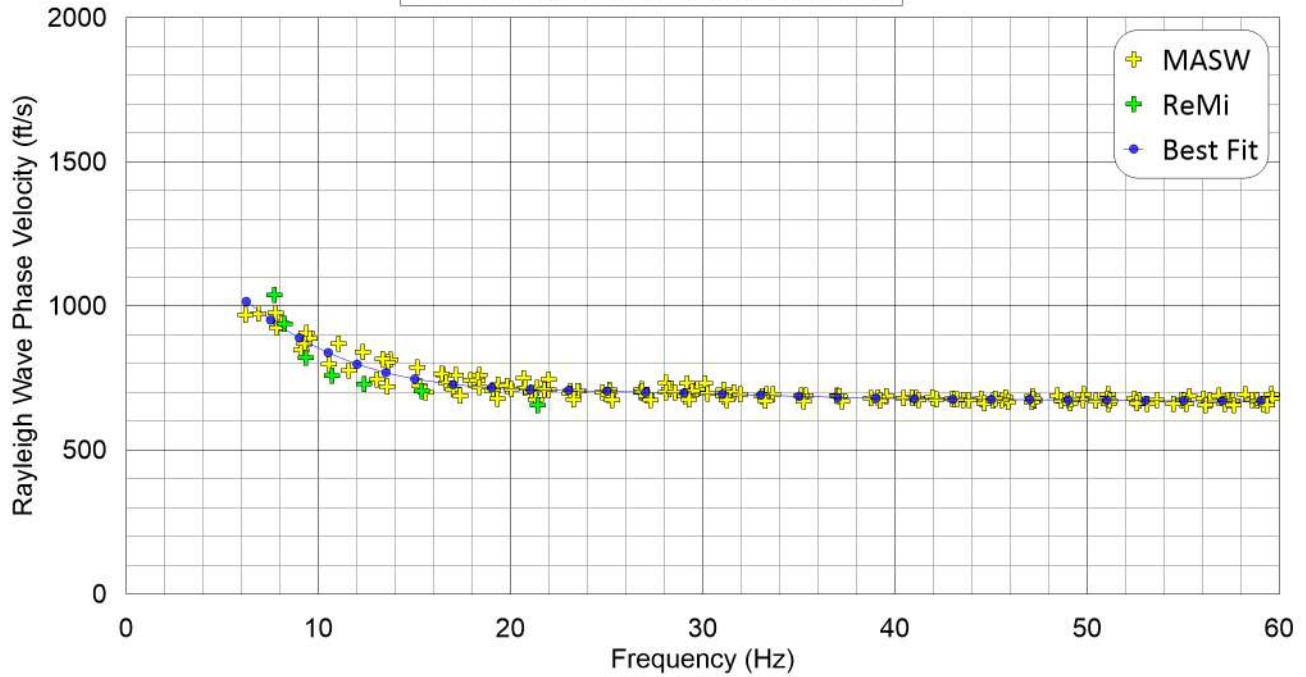


REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL

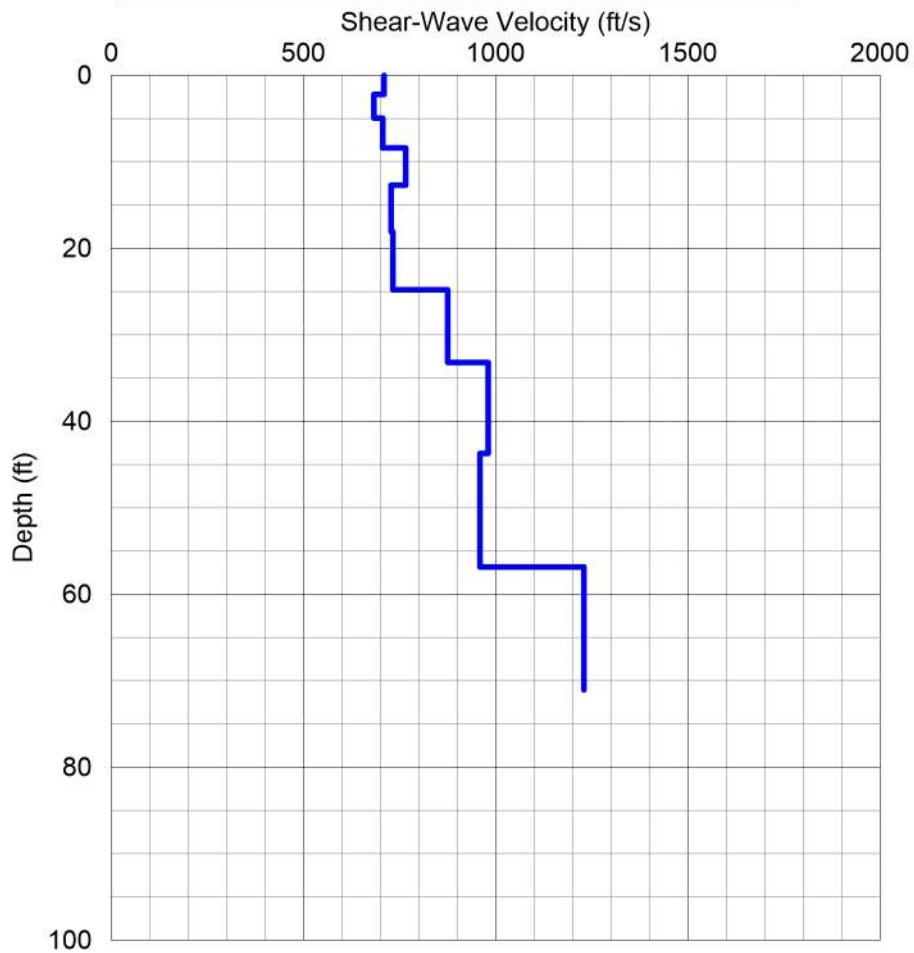


SW21-3: S-WAVE VELOCITY COMBINED SOURCE MODEL

SURFACE WAVE DISPERSION CURVE

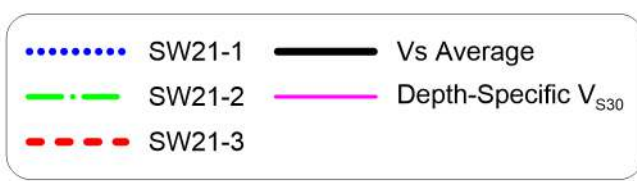
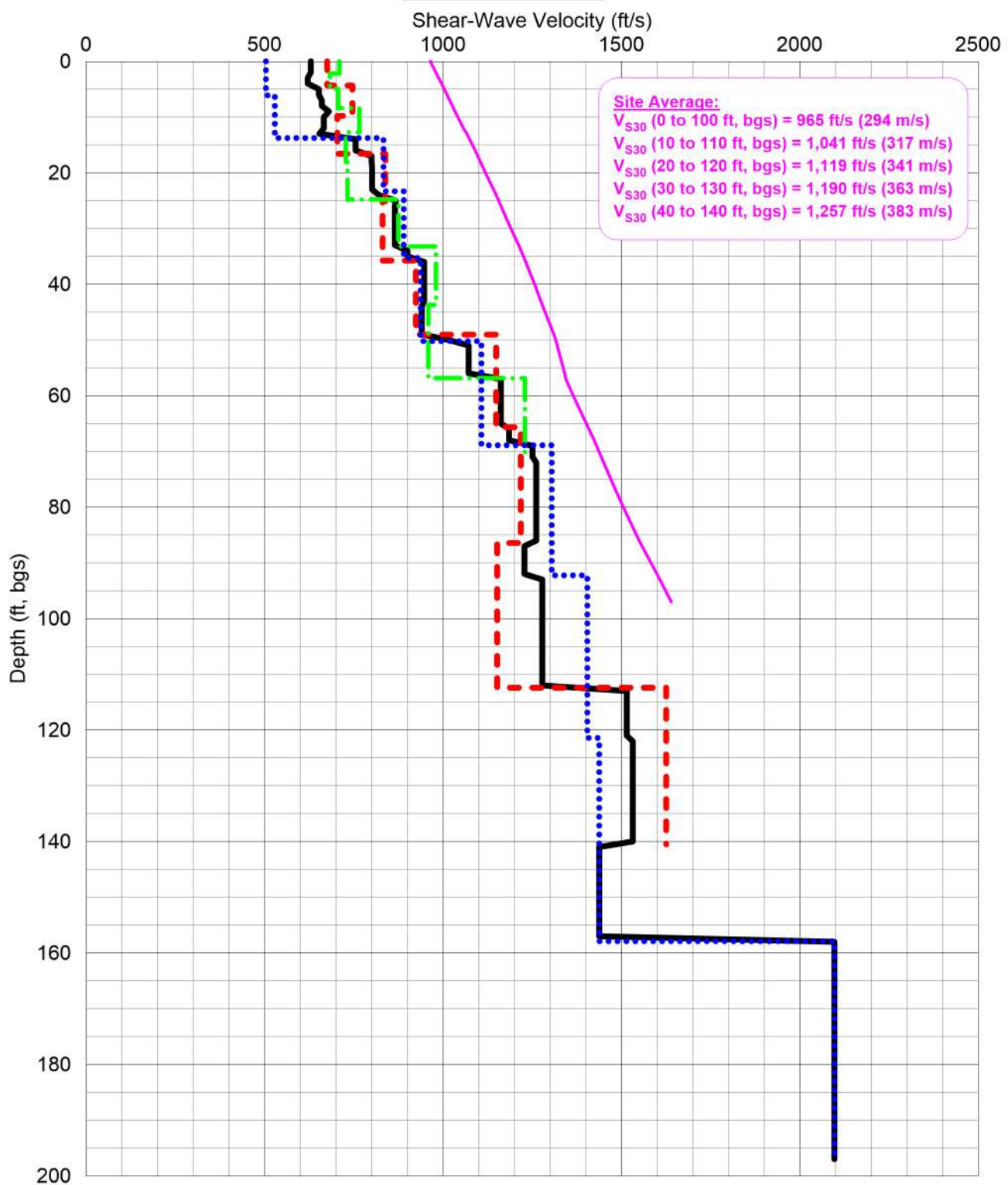


REPRESENTATIVE SHEAR-WAVE VELOCITY MODEL



SW21-2: S-WAVE VELOCITY COMBINED SOURCE MODEL

1056 La Cienega



SHEAR WAVE VELOCITY PROFILE SUMMARY

APPENDIX F

GROUND MOTION ANALYSIS

F.1 INTRODUCTION

This Appendix presents the ground motion evaluation results for the subject site located on Figure F-1 in Los Angeles, California. Specifically, this Appendix contains the recommended site-specific response spectra. This Appendix will be updated with the earthquake time history analysis results as the structural design progresses.

The currently proposed development includes the design and construction of 27-story highrise tower that includes three parking levels (one subterranean and two above ground) and an amenity deck. The estimated fundamental spectral period of interest of the structure is not finalized at this time, but it is estimated to be about 2.0- to 3.0-seconds, and will be confirmed when the structural design is finalized.

We understand that the design for this structure is being carried out in conformance with the 2019 California Building Code (CBC 2019) and ASCE 7-16 requirements using the performance-based design procedure specified by the 2021 Los Angeles Tall Buildings Structural Design Council (LATBSDC). To meet the performance-based design requirements, two levels of seismic evaluation will be completed: [1] a Serviceability Evaluation and [2] a Collapse Prevention Evaluation. The Serviceability Evaluation will be performed using the Service Level Earthquake (SLE) response spectrum, and the Collapse Prevention Evaluation will be performed using the Risk-Targeted Maximum Considered Earthquake (MCE_R) response spectrum. The design of nonstructural components might be based on the Design Response Spectrum (DRS), which is included for completeness.

To fulfill the seismic design requirements, the following site-specific response spectra are developed herein and summarized in this Appendix.

- “Maximum Considered Earthquake” uniform hazard spectrum (also known as the MCE_R response spectrum); This response spectrum is based on risk-targeted, maximum-rotated ordinates at 5% damping and corresponds to a 1% probability of collapse in a 50-year period.
- “Service-Level Earthquake” uniform hazard spectrum (also known as the SLE response spectrum); This response spectrum is based on average horizontal spectral ordinates at 2.5% damping and corresponds to a 50% probability of exceedance in a 30-year period.
- “Design-Level Earthquake” uniform hazard spectrum (also known as a DLE or DBE response spectrum, or DRS). This spectrum is based on maximum-rotated ordinates at 5% damping and corresponds to 2/3 of the MCE_R response spectrum.

The Collapse Prevention Evaluation also requires the development of eleven pairs of earthquake time histories scaled or spectrally matched to the site-specific Maximum Considered Earthquake (MCE_R) response spectrum, in accordance with the requirements of Section 16.2 of ASCE 7-16. Because this project will be subject to the performance-based peer review process, the seed acceleration time histories selected for the Collapse Prevention Evaluation will be reviewed and approved by the Peer Review Panel prior to performing the spectral matching. The final matched acceleration time histories to be used in the nonlinear response analysis will be documented upon receiving approval of the site-specific response spectra and seed time histories by the structural engineering team and the review panel. Note that if the site location or site conditions change appreciably, the ground motion results presented herein would need to be re-evaluated.

F.2 SEISMIC SITE CHARACTERIZATION

The seismic site characterization for this study consisted of defining the site parameters needed to account for soil non-linearity in ground motion attenuation models. The shear wave velocity in the upper 30 meters of the site (V_{S30}) is the primary parameter used to approximate soil non-linearity in the ground motion models. The remaining site parameters in the ground motion attenuation models are the basin terms $Z_{1.0}$ and $Z_{2.5}$, which represent the depth to the 1.0 km/s and 2.5 km/s shear wave velocities, respectively.

As part of this evaluation, shear wave velocity measurements were collected at the site using Multichannel Analysis of Surface Waves (MASW) and Refraction Microtremor (ReMi) methods along three survey lines. The results and more information on the geophysical methods and analysis procedures is provided in Appendix E of this report. On Figure F-2, the V_{S30} values are calculated for a range of depths (0 to 40-ft) below existing ground surface. The data are presented in this format to allow for efficient interpretation of the V_{S30} value at a particular outcropping depth, as well as to provide information on the sensitivity of the V_{S30} to the shallow soils. The V_{S30} values are calculated per ASCE 7-16, Section 20.4.1.

Based on information from SEOR, we understand that the proposed structure consists of one basement on a mat foundation. Furthermore, it is our understanding that the majority of the seismic loading will be accommodated by the foundation and that lateral loading on the basement walls of the structure is minimal; therefore, the soils at and below the foundation level are expected to control the seismic input. In accordance with the structural properties and Section 3.2.4 of the 2021 LATBSDC guidelines, we recommend the V_{S30} be computed from the 10-ft depth. This corresponds to a V_{S30} value of 1,040 ft/s (317 m/s). This V_{S30} value corresponds to Site Class D ($600 < V_{S30} < 1,200$ ft/s) in ASCE 7-16.

The remaining site parameters in the ground motion attenuation models are the basin terms $Z_{1.0}$ and $Z_{2.5}$. The approximate depths to these interfaces were estimated to be 460 m and 5.1 km, respectively. These estimates were based on the SCEC Community Velocity Model (CVM-S4) by Magistrale et al. (2000 and 2012) and are in general agreement with values previously used for projects in the vicinity.

F.3 CODE-BASED VALUES

Given the site latitude and longitude (located near 118. 375974°W, 34.057089°N) and estimated shear wave velocity, mapped seismic hazard values were queried from the SEAOC/OSHPD Seismic Design Maps Tool application online at <https://seismicmaps.org/>. As discussed above in Section F.2 of this Appendix, the estimated V_{S30} at the site foundation level is 1,040 ft/s (317 m/s). This V_{S30} value corresponds to site classification for seismic design of Site Class D ($600 < V_{S30} < 1,200$ ft/s). The mapped design parameters below are based on this information.

The general procedure ground motion analysis carried out in accordance with Chapter 16A of the 2019 CBC and Section 11.4.4 of ASCE 7-16 results in mapped acceleration parameters S_s and S_1 of 2.068 g and 0.737 g, respectively, and site amplification factors F_a and F_v of 1.0 and 2.5, respectively. The general design spectral acceleration parameters S_{D5} and S_{D1} are 1.379 g and 1.228 g, respectively, and Seismic Design Category D for Risk Category II structures. The S_{D5} and S_{D1} values are superseded by the site-specific values presented in this Appendix but have been provided here for completeness.

F.4 SEISMIC HAZARD ANALYSIS

Probabilistic and Deterministic Seismic Hazard Analyses (PSHA and DSHA, respectively) involve the characterization of seismic sources, transmission paths for seismic energy, and the local site conditions. Seismic sources pertinent to ground motion hazards at the site are characterized based on geologic information. The effects of transmission paths and local site conditions are estimated with ground motion attenuation relationships, which provide the variation in peak horizontal and/or vertical acceleration or spectral acceleration with distance for a given local site condition. Key information on the computational platforms, seismic sources, and attenuation relationships used in this study is summarized below, followed by the results of the PSHA and DSHA. The resulting response spectra are presented in the following section (Section F.5) of this Appendix.

The site is located within a seismically active region of southern California, as evidenced by Quaternary faulting and historic earthquakes. The locations of Quaternary-active surface-rupturing faults mapped

by the US Geological Survey (USGS, 2018) and instrumentally-recorded earthquakes (Hauksson et al., 2018) relative to the project site are shown on Figure F-3a. The closest Late Quaternary (within the last 15,000 years) surface fault ruptures occurred on the Newport-Inglewood Fault (approximately 2 west of the site) and the Hollywood Fault (approximately 4 km north of the site).

The epicenter for the 1994 Northridge earthquake was approximately 27 km northwest of the project site. Based on nearby recording stations in the NGA/PEER database (LA - Saturn St, SSN: 584), the event produced peak horizontal ground accelerations (PGA) and peak ground velocities (PGV) of about 0.44 g and 39.5 cm/s, respectively, near the project site.

The Seismic Source Characterization (SSC) models used for this project are based on the characterization used by the USGS to develop the 2014 version of National Seismic Hazard Maps (NSHM; Petersen et al., 2014). Both discrete faults and background sources were included. This model includes the Uniform California Earthquake Rupture Forecast version 3 (UCERF3; by WGCEP, 2013) branch average models (i.e., both alternatives) for discrete crustal faults and gridded background seismicity. The locations of the seismic sources relative to the project site, as implemented in the PSHA, are shown on the fault map on Figure F-3b. The best-estimate parameters (including maximum magnitude, closest distance, slip rate, and style of faulting) for these seismic sources are summarized in Table F-1; specific scenarios evaluated for the DSHA are presented in Table F-2.

Seismic shaking is estimated using empirical ground motion attenuation relationships and calculated as the pseudo-spectral acceleration (SA) for a given period. Calculated values represent the average horizontal component considering 5% damping. Four of the five of the Next Generation Attenuation West 2 (NGA W2) ground motion attenuation models were used in the PSHA to calculate the horizontal response spectra: Abrahamson et al., 2014; Boore et al., 2014; Campbell and Bozorgnia, 2014; and Chiou and Youngs, 2014. The Idriss (2014) model was not used based on the V_{S30} for the site and the applicability criteria for the model. Each of the attenuation relationships was assigned an equal weight of 1/4 to approximately address the “modeling” part of the epistemic uncertainty.

A site-specific Probabilistic Seismic Hazard Analysis (PSHA) was completed to generate hazard curves and equal-hazard response spectra at the site for the Maximum Considered Earthquake (i.e., the MCE_R) and the Service-Level Earthquake (SLE). The basic results of the PSHA are presented in terms of seismic hazard curves, which show the annual probability of exceedance of a given spectral acceleration (SA), including horizontal and/or vertical peak ground acceleration (PGA). The annual probability of exceedance is based on the calculated mean number of events per year that result in the spectral acceleration being exceeded at the site. Deaggregation plots are also useful for presenting PSHA results for a specified average return period (ARP) and SA; they show the percentage

contribution to the total site seismic hazard based on distance and magnitude. Finally, equal-hazard spectra are used to identify a uniform hazard level (i.e., a specified ARP) over a range of periods.

Figures F-4a and F-4b present seismic hazard curves for horizontal PGA and the 3.0-second period. The total hazard (solid black line) and the contributions of various seismic sources to the total seismic hazard are shown. As indicated on Figure F-4a, the Santa Monica Fault and Newport-Inglewood are the major contributors with the Puente Hills System (Puente Hills Alt., Puente Hills LA, Puente Hills Coyote Hills, and Puente Hills Santa Fe Springs faults) also providing contribution to the PGA hazard for ARPs longer than about 500 years. At shorter return periods, background seismicity controls the PGA hazard. The 3.0-second hazard contribution (Figure F-4b) is also controlled by the the Santa Monica Fault and Newport-Inglewood at ARPs longer than about 500 years with the San Andreas controlling the short return periods.

Magnitude-distance deaggregations for PGA and 3.0-seconds were also evaluated for the following ARPs: 43-yr (50% probability of exceedance in 30 years), and 2,475-yr (2% probability of exceedance in 50 years). The deaggregation plots are shown on Figures F-5a and F-5b. The vertical axis of the plots show the relative intensity of the magnitude-distance contribution with respect to the epsilon value (number of standard deviations above or below the median). Epsilon values of ± 1 correspond to the 16th/84th percentiles; values of ± 2 indicate 2nd/98th percentiles; and an epsilon value of zero is the median or 50th percentile.

As shown on Figure F-5a, the 2,475-yr PGA hazard is controlled by MW 6.0 to 7.5 earthquakes located within 20 km of the site that produce median to 98th percentile ground motions. These magnitude-distance bins correspond to characteristic events on several sources, including the Santa Monica, Newport-Inglewood, Puente Hills (Alt 1. and LA), Elysian Park (Upper), Compton, Hollywood, and Raymond. The 43-yr PGA hazard is controlled background events MW 5.0 to 7.0 withing approximately 40 km of the site; however, contribution from characteristic earthquakes on the San Andreas System 56 km away is also visible.

Figure F-5b presents the deaggregation for the 3.0-second period. The 2,475-yr ground motions are controlled by characteristic events on several sources, including the Santa Monica, Newport-Inglewood, Puente Hills (Alt 1. and LA), and Compton Faults. The 43-yr 3.0-second hazard deaggregation is also similar, except the San Andreas contributions are significantly higher.

The results of the PSHA at periods between 0.01 and 10 seconds were aggregated into a 2,475-yr ARP uniform hazard spectrum, as shown on Figure F-6. The probabilistic MCE_R spectrum, which represents the maximum rotated, risk-targeted ordinates per ASCE 7-16, is also shown on Figure F-6. The ordinates are tabulated in Table F-3 in Column 6. This spectrum was developed using one set of scale

factors to adjust the calculated ordinates (which are the average horizontal component of ground motion) to the maximum rotated component of ground motion, and a second set of scale factors was used to adjust the ordinates from hazard representing 2% probability of exceedance in 50 years (the 2,475-yr ARP) to risk, which represents a 1% probability of collapse in 50 years. The adjustment between average horizontal and maximum rotated component is based on the period-specific ratios in Shahi and Baker (2014). The adjustment between the hazard and risk-targeted ordinates is based on the mapped ratios provided by ASCE 7-16 Method 1 (21.2.1.1). At the site latitude and longitude, a scale factor of 0.897 is specified for periods 0.2-second and shorter and a scale factor of 0.897 is used for periods of 1.0-second and longer; scale factors for periods between 0.2- and 1.0-second are linearly interpolated. Both of these scale factors are incorporated in the probabilistic MCE_R spectrum shown on Figure F-6, and the process of developing the probabilistic MCE_R spectral ordinates is shown on Table F-3 in Columns 3 through 6.

The Serviceability Evaluation per the 2020 LATBSDC guidelines uses the Service-Level Earthquake (SLE) spectrum, which based on a uniform hazard spectrum reflecting ground motions with a 50% probability of exceedance in 30 years (43-year return period). Accordingly, the results of the horizontal PSHA at periods between 0.01 and 10 seconds are also aggregated into a 43-yr ARP uniform hazard spectrum on Figure F-6. Development of the SLE spectrum, including conversion of the hazard ordinates to the target damping ratio, is discussed below.

A deterministic seismic hazard analysis (DSHA) was performed for the site following the guidelines provided in ASCE 7-16. Based on the seismic source characterization and the results of the PSHA, several faults were evaluated for the DSHA. Table F-2 lists the key contributors to the DSHA ground motions, as well as the fault parameters used in the analysis. The DSHA scenarios were evaluated using the ground motion models and site parameters defined above.

Uniform hazard spectra for the key DSHA scenarios in Table F-2 are shown on Figure F-7. The DSHA ordinates reflect the 84th percentile maximum rotated component of ground motion. The modification from the average horizontal component of ground motion to the maximum rotated component was performed using the same methodology described above for the development for the probabilistic MCE_R (i.e., the Shahi and Baker, 2014 period-specific ratios).

The code-based minimum, developed in accordance with of ASCE 7-16 Supplement 1, Section 21.2.2 is also shown on Figure F-7. As shown on Figure F-7, the Compton scenario controls the deterministic MCE_R spectrum at periods shorter than 1.5-seconds. At longer periods, the Newport-Inglewood scenarios with Directivity controls. The deterministic MCE_R spectral ordinates are tabulated in Table F-3 in Column 10, and the process of developing the deterministic MCE_R spectral ordinates is shown in Table F-3 in Columns 7 through 10.

F.5 SITE-SPECIFIC RESPONSE SPECTRA

It is our understanding that the structural evaluation is being carried out in conformance with the 2019 CBC requirements and ASCE 7-16 requirements for performance-based design, using the procedure specified by the 2020 LATBSDC guidelines. Accordingly, two levels of seismic evaluation are required for this project: Serviceability Evaluation and Collapse Prevention Evaluation. The Serviceability Evaluation uses the Service-Level Earthquake (SLE) spectrum, which is represented by a uniform hazard spectrum reflecting ground motions with a 50% probability of exceedance in 30 years (43-yr return period) with a reduced damping ratio (< 5%). The Collapse Prevention Evaluation uses the site-specific MCE_R response spectrum, developed in accordance with the requirements of Section 21.2 of ASCE 7-16. For completeness, the site-specific DRS, developed in accordance with the requirements of Section 21.3 of ASCE 7-16, is also provided for the design of non-structural components. The December 2018 ASCE 7-16 Supplement 1 was followed in developing both the site-specific MCE_R and DRS spectra.

Figure F-8 shows the final development of the site-specific MCE_R response spectrum. The final MCE_R is developed as the lesser of the deterministic MCE_R and the probabilistic MCE_R response spectra (per ASCE 7-16, Section 21.2.3), but no less than the code-based minimum (per ASCE 7-16, Supplement 1, Section 21.2.3).

As shown on Figure F-8, the probabilistic MCE_R spectrum controls at all periods; however, the final site-specific MCE_R is adjusted such that none of the spectral ordinates fall below the code-based minimum for the period range between 2.0-seconds and longer. The final site-specific MCE_R spectrum is shown highlighted on Figure F-8, and the spectral ordinates are tabulated in Table F-3, Column 12. The process of developing the site-specific MCE_R spectral ordinates is shown in Table F-3 in Columns 6 and 10 through 12.

The Design Response Spectrum (DRS) was developed as 2/3 of the site-specific MCE_R , but no less than the code-based minimum (which is defined as 80% of the code-based spectrum using ASCE 7-16, Section 11.4.6). The process of developing the DRS is shown on Figure F-9. The final recommended DRS is shown highlighted on Figure F-9, and the ordinates are tabulated in Table F-4, Column 3.

The SLE response spectrum, which is based on the 43-year ARP uniform hazard spectrum, is shown on Figure F-6. The SLE response spectrum represents a 50% probability of exceedance in 30 years at a reduced damping ratio (< 5%). Based on communications from the SEOR, a critical damping value of 2.5% is used in the SLE development. Specifically, the 43-year ARP uniform hazard spectrum ordinates were converted from 5% spectral damping (as is predicted by the GMPEs in the hazard calculation) to

2.5% damping using the empirically-based Damping Scaling Factor (DSF) relationship in Rezaeian et al. (2012). This model uses magnitude and distance as parameters to estimate period-specific DSFs. The mean magnitude and distance for each spectral ordinate at the 43-yr ARP were used in the DSF calculation. The final recommended SLE is tabulated in Table F-4 in Column 6. The process of developing the SLE ordinates is also shown in Table F-4.

Using ASCE 7-16, Section 21.4, the site-specific seismic design parameters for new structures at the project site are defined below. These parameters were developed in accordance with ASCE 7-16, Section 21.3.

- $S_{DS} = 1.563$ g, based on 90% of the spectral acceleration at a period of 0.3-seconds
- $S_{D1} = 1.093$ g, based on the spectral acceleration at a period of 1.0-second
- $S_{MS} = 2.345$ g, based on 1.5 times S_{DS}
- $S_{M1} = 1.640$ g, based on 1.5 times S_{D1}

F.6 REFERENCES

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**TABLE F-1
CHARACTERIZATION⁽¹⁾ OF FAULTS SIGNIFICANT TO THE
1056 LA CIENEGA BLVD.**

Fault Name	Style of Faulting ⁽²⁾	Maximum Magnitude (Mw)	Slip Rate (mm/yr)	Closest Rupture Distance From Site (km)	Fault Name	Style of Faulting ⁽²⁾	Maximum Magnitude (Mw)	Slip Rate (mm/yr)	Closest Rupture Distance From Site (km)
San Vicente	RV	6.1	0.2	1.9	Clamshell-Sawpit	RV	6.4	0.3	38
Newport-Inglewood	SS	7.2	1.2	2.0	Holser	RV	6.7	0.5	39
North Salt Lake	RV	5.8	0.1	3.8	Simi-Santa Rosa	OBL	6.8	1.1	43
Hollywood	OBL	6.5	1.3	4.0	San Jose	OBL	6.5	0.3	45
Santa Monica	OBL	6.7	1.1	4.1	Richfield	RV	6.1	0.2	46
Puente Hills (LA)	RV	6.8	0.6	5.1	Peralta Hills	RV	6.4	0.3	48
Puente Hills	RV	6.9	1.7	8.8	Oak Ridge (Onshore)	RV	7.1	2.6	48
Elysian Park (Lower)	RV	6.8	0.1	11	Yorba Linda	RV	6.3	0.1	49
Compton	RV	7.3	0.8	12	Del Valle	RV	6.2	1.0	49
Elysian Park (Upper)	RV	6.5	1.4	12	Malibu Coast (Extension)	OBL	6.9	0.5	52
San Pedro Escarpment	RV	7.1	0.2	12	Chino	OBL	6.7	0.9	53
Malibu Coast	OBL	6.9	0.8	14	San Joaquin Hills	RV	6.8	0.5	55
Anacapa-Dume	OBL	7.1	0.7	17	San Cayetano	RV	7.1	2.9	58
Raymond	OBL	6.6	1.3	17	Cucamonga	RV	6.8	1.7	59
Palos Verdes	SS	7.4	2.3	17	Sisar	RV	6.8	0.8	62
Verdugo	RV	6.8	0.6	20	San Andreas ⁽³⁾	SS	8.2	29.0	63
Santa Monica Bay	RV	6.8	0.1	21	Newport-Inglewood Offshore	SS	7	1.0	65
Northridge Hills	RV	6.8	1.0	24	Santa Cruz-Catalina Ridge	OBL	7.4	1.1	65
Redondo Canyon	RV	6.6	0.4	24	San Diego Trough North	SS	7.3	1.6	67
Northridge	RV	6.9	1.3	26	Fontana	SS	6.6	0.3	72
Puente Hills (Santa Fe Springs)	RV	6.4	0.8	26	Ventura-Pitas Point	OBL	7.1	1.5	72
Mission Hills	RV	6.3	0.8	26	Santa Ynez (East)	SS	7.2	1.5	76
Santa Susana East (connector)	RV	6.2	1.9	26	Pine Mountain	RV	7.2	0.3	76
Sierra Madre	RV	7.2	1.5	27	Oceanside	RV	7.2	0.7	77
Sierra Madre (San Fernando)	RV	6.5	1.6	29	San Jacinto ⁽³⁾	SS	7.9	6.0	80
Elsinore - Whittier ⁽³⁾	SS	7	4.2	30	Santa Cruz Island	OBL	7.2	0.9	82
Puente Hills (Coyote Hills)	RV	6.7	0.8	31	Channel Islands Thrust	RV	7.2	1.0	82
Santa Susana	RV	6.9	3.2	32	Oak Ridge (Offshore)	RV	6.9	1.7	84
San Gabriel (Extension)	SS	7.1	0.5	32	Mission Ridge-Arroyo Parida-Santa Ana	RV	7.0	1.1	87
Anaheim	RV	6.3	0.1	33	Cleghom	SS	6.7	0.5	89
San Gabriel	OBL	7.3	0.6	33	Red Mountain	RV	7.4	2.2	92
San Pedro Basin	SS	7.1	1.1	35	Channel Islands Western Deep Ramp	RV	7.2	0.4	95

Notes:

(1) Source characterization based on information published by SCEC/USGS UCERF2 (WGCEP, 2008), 2008 NSHM (Petersen et al., 2008), and UCERF3 (WGCEP, 2013a,b).

(2) SS=Strike-Slip, OBL=Oblique, RV=Reverse or Thrust, NOR=Normal.

(3) Characterization used a distribution of magnitude and slip rates; best estimate for deterministic case shown.

**TABLE F-2
DETERMINISTIC SEISMIC HAZARD ANALYSIS FAULT CHARACTERIZATION
1056 LA CIENEGA BLVD.**

<i>Fault</i>	Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9	Column 10	Column 11	Column 12
	M_W	F_{RV}	F_N	F_{HW}	Z_{TOR}	Z_{BOT}	<i>Dip</i>	<i>W</i>	Z_{HYP}	R_{RUP}	R_{JB}	R_X
Newport-Inglewood Onshore	7.4	0	0	0	0	15	90	15.0	10.2	2.0	2.0	2.0
Puente Hills (LA)	6.8	1	0	0	2.1	15	27	28.4	7.8	5.1	4.2	-4.2
San Vicente	6.1	1	0	0	1.6	17	66	16.9	7.0	1.9	1.9	-1.9
Santa Monica-Hollywood	7	1	0	0	0	17.3	70	18.4	10.2	4.1	4.1	-4.1
Puente Hills (Alt1)	7	1	0	0	5	13	25	18.9	10.2	8.8	8.0	-8.0
Compton	7.3	1	0	1	5.2	15	20	28.7	9.4	12.4	0.0	21.2
Palos Verdes	7.4	0	0	0	0	13.6	90	13.6	10.2	19.2	19.2	19.2
Whittier-Elsinore	7	0	0	0	0	15	90	15.0	10.2	30.2	30.2	30.2
San Andreas	8.2	0	0	0	0	15	90	15.0	10.2	62.8	62.8	62.8

Key

Column 1	= Moment magnitude.
Column 2	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique, thrust.
Column 3	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal.
Column 4	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise.
Column 5	= Depth to top of coseismic rupture (km).
Column 6	= Depth to bottom of the seismogenic crust (km).
Column 7	= Average dip of rupture plane (degrees).
Column 8	= Fault rupture width (km).
Column 9	= Hypocentral depth from the earthquake (km), based on Campbell and Bozorgnia (2014) model.
Column 10	= Closest distance to coseismic rupture (km).
Column 11	= Closest distance to surface projection of coseismic rupture (km).
Column 12	= Horizontal distance from top of rupture measured perpendicular to fault strike (km).

**TABLE F-3
SITE-SPECIFIC FREE-FIELD MCE_R DEVELOPMENT CALCULATION SHEET
1056 LA CIENEGA GROUND-MOTION EVALUATION, FOUNDATION LEVEL**

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8	Column 9	Column 10	Column 11	Column 12
Period	Frequency	2475-yr UHS (PSHA)	Risk Collapse Scaling Factors	Max. Orientation Scaling Factors	Probabilistic MCE _R	84th %tile DSHA	Max. Direction 84th %tile DSHA	Code-Based Deterministic Minimum MCE _R	Deterministic MCE _R	Code Minimum MCE _R	Final Site-Specific MCE _R
		RotD50		RotD50	RotD100	RotD50	RotD100	RotD100	RotD100	RotD100	RotD100
(sec)	(Hz)	(g)	-	-	(g)	(g)	(g)	(g)	(g)	(g)	(g)
0.010	100	0.946	0.897	1.190	1.010	1.137	1.353	0.681	1.353	0.717	1.010
0.020	50	0.950	0.897	1.190	1.014	1.144	1.361	0.685	1.361	0.773	1.014
0.030	33	0.985	0.897	1.190	1.051	1.171	1.393	0.701	1.393	0.829	1.051
0.050	20	1.138	0.897	1.190	1.214	1.310	1.559	0.784	1.559	0.941	1.214
0.075	13	1.434	0.897	1.190	1.531	1.568	1.866	0.938	1.866	1.080	1.531
0.100	10	1.690	0.897	1.190	1.804	1.810	2.153	1.083	2.153	1.219	1.804
0.150	6.67	1.979	0.897	1.200	2.130	2.119	2.543	1.279	2.543	1.498	2.130
0.200	5.00	2.149	0.897	1.210	2.333	2.411	2.917	1.467	2.917	1.654	2.333
0.250	4.00	2.280	0.897	1.220	2.495	2.598	3.170	1.594	3.170	1.654	2.495
0.300	3.33	2.381	0.897	1.220	2.605	2.811	3.429	1.725	3.429	1.654	2.605
0.400	2.50	2.338	0.897	1.230	2.580	2.909	3.578	1.800	3.578	1.654	2.580
0.500	2.00	2.225	0.897	1.230	2.455	2.818	3.466	1.743	3.466	1.654	2.455
0.750	1.33	1.837	0.897	1.240	2.043	2.355	2.920	1.469	2.920	1.654	2.043
1.000	1.00	1.475	0.897	1.240	1.640	1.833	2.272	1.143	2.272	1.474	1.640
1.500	0.67	0.961	0.897	1.240	1.069	1.278	1.584	0.797	1.584	0.983	1.069
2.000	0.50	0.691	0.897	1.240	0.769	0.986	1.222	0.615	1.222	0.737	0.769
3.000	0.33	0.411	0.897	1.250	0.461	0.679	0.849	0.427	0.849	0.491	0.491
4.000	0.25	0.267	0.897	1.260	0.302	0.490	0.617	0.310	0.617	0.369	0.369
5.000	0.20	0.190	0.897	1.260	0.215	0.366	0.461	0.232	0.461	0.295	0.295
7.500	0.13	0.101	0.897	1.280	0.116	0.192	0.246	0.124	0.246	0.197	0.197
10.000	0.10	0.062	0.897	1.290	0.072	0.120	0.155	0.078	0.155	0.118	0.118

Note: Significant figures are provided for computational purposes only and do not necessarily reflect accuracies to those significant figures.

Key

Column 1	= Spectral period in seconds.
Column 2	= Spectral frequency (inverse of spectral period) in Hertz.
Column 3	= Mean uniform hazard spectral ordinates for 2,475- yr average return period in units of g for 5% damping; GMRot150 and RotD50 are produced by NGA West 1 and West2, respectively.
Column 4	= Site-specific risk coefficient (C _R) from USGS.
Column 5	= Scale factor to obtain maximum-oriented spectral acceleration; from Shahi and Baker (2014).
Column 6	= Probabilistic risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 7	= 84th percentile deterministic hazard spectral ordinates in units of g for 5% damping; ordinates are maximum of all deterministic scenarios, therefore spectrum may not represent a single event.
Column 8	= Deterministic, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 9	= Code-based (ASCE 7-16 Supplement 1, Ch. 21.2.2) deterministic lower limit for risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 10	= Deterministic maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping; maximum value from Columns 8 and 9.
Column 11	= 80% of code-based (ASCE 7-16, Ch. 11) risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping.
Column 12	= Final risk-targeted, maximum considered earthquake ground motion spectral ordinates in units of g for 5% damping; minimum value from Columns 6 and 10, but no less than Column 11.

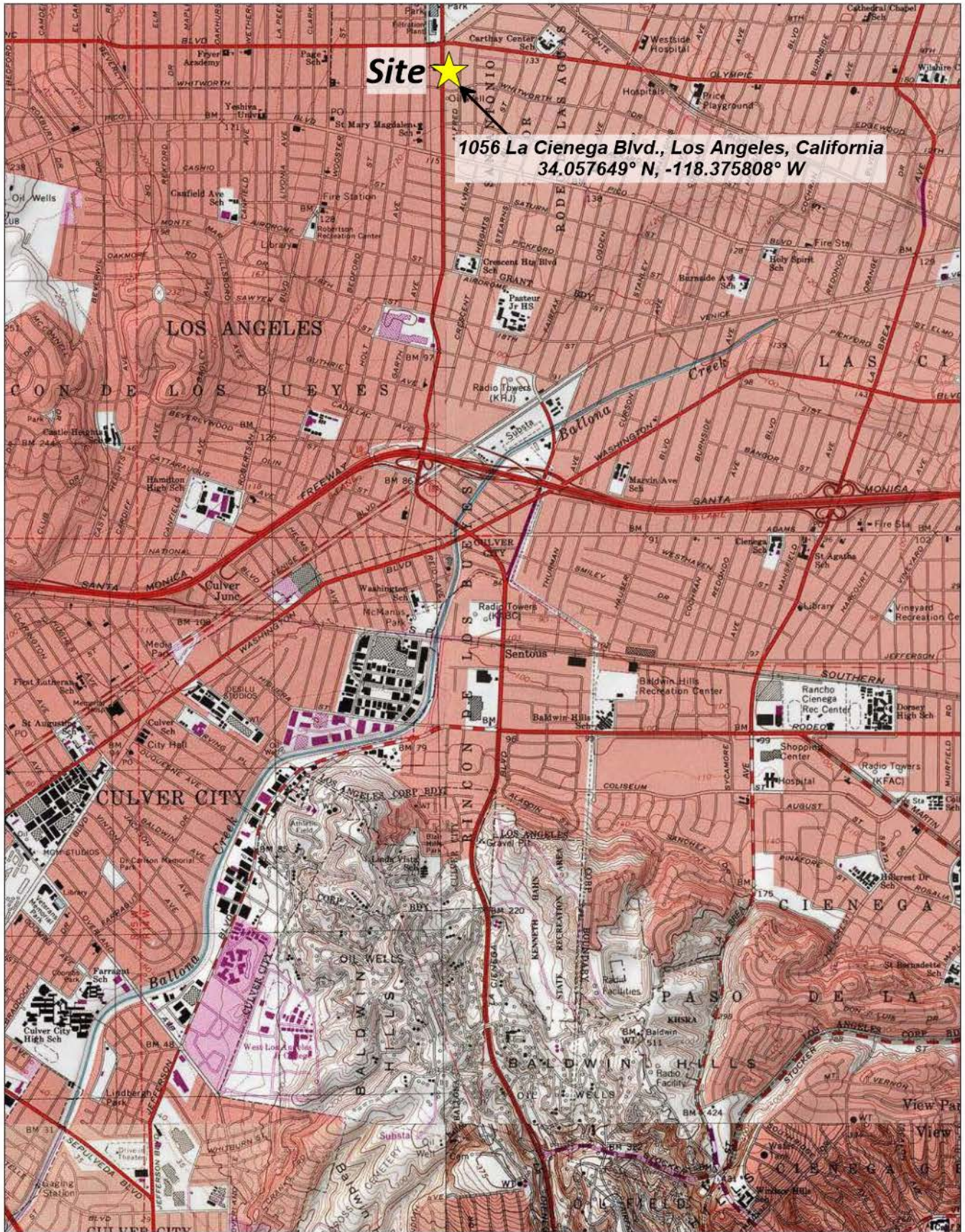
**TABLE F-4
SITE-SPECIFIC DRS AND SLE DEVELOPMENT CALCULATION SHEET
1056 LA CIENEGA GROUND-MOTION EVALUATION, FOUNDATION LEVEL**

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6
Period	Frequency	<i>Final Site-Specific DRS</i>	<i>43-yr UHS (PSHA)</i>	<i>Damping Scaling Factors</i>	<i>SLE @ 2.5% Damping</i>
		<i>RotD100</i>	<i>RotD50</i>		<i>RotD50</i>
(sec)	(Hz)	(g)	(g)	-	(g)
0.01	100	0.673	0.187	1.000	0.187
0.02	50	0.676	0.187	1.005	0.188
0.03	33	0.701	0.195	1.023	0.200
0.05	20	0.810	0.227	1.074	0.243
0.075	13	1.021	0.287	1.136	0.326
0.1	10	1.203	0.344	1.183	0.407
0.15	6.67	1.420	0.419	1.220	0.511
0.2	5.00	1.555	0.448	1.237	0.554
0.25	4.00	1.663	0.454	1.236	0.562
0.3	3.33	1.737	0.445	1.241	0.553
0.4	2.50	1.720	0.397	1.244	0.494
0.5	2.00	1.636	0.355	1.244	0.442
0.75	1.33	1.362	0.258	1.238	0.319
1	1.00	1.093	0.192	1.236	0.237
1.5	0.67	0.713	0.121	1.232	0.150
2	0.50	0.512	0.085	1.223	0.104
3	0.33	0.328	0.051	1.219	0.062
4	0.25	0.246	0.034	1.208	0.041
5	0.20	0.197	0.024	1.202	0.029
7.5	0.13	0.131	0.012	1.182	0.015
10	0.10	0.079	0.007	1.135	0.008

Note: Significant figures are provided for computational purposes only and do not necessarily reflect accuracies to those significant figures.

Key

Column 1	= Spectral period in seconds.
Column 2	= Spectral frequency (inverse of spectral period) in Hertz.
Column 3	= Final DRS ground motion spectral ordinates in units of g for 5% damping; 2/3 of MCER.
Column 4	= Mean uniform hazard spectral ordinates for 43- yr average return period in units of g for 5% damping.
Column 5	= Damping Scaling Factor used to convert spectral ordinates from 5% damping; developed per Rezaeian et al. (2012).
Column 6	= Service-Level Earthquake ground motion spectral ordinates in units of g for reported damping; developed per Rezaeian et al. (2012).



SITE LOCATION MAP

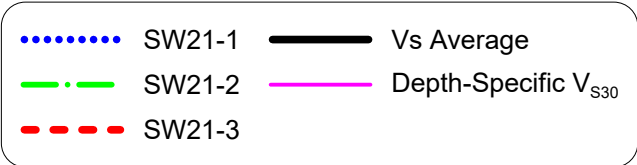
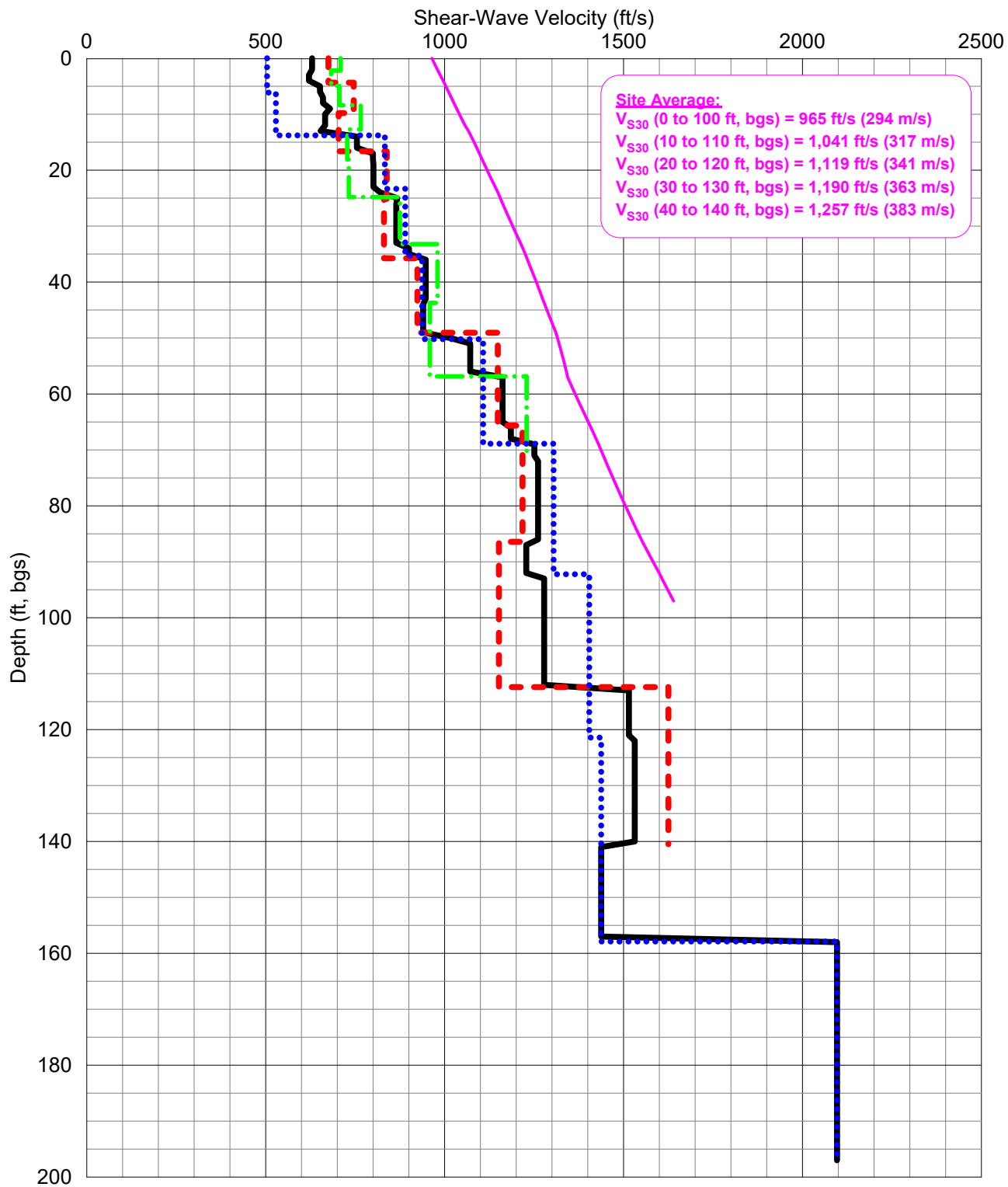
Project No.: 21086A

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Date: DEC 2021

Figure F-1

1056 La Cienega



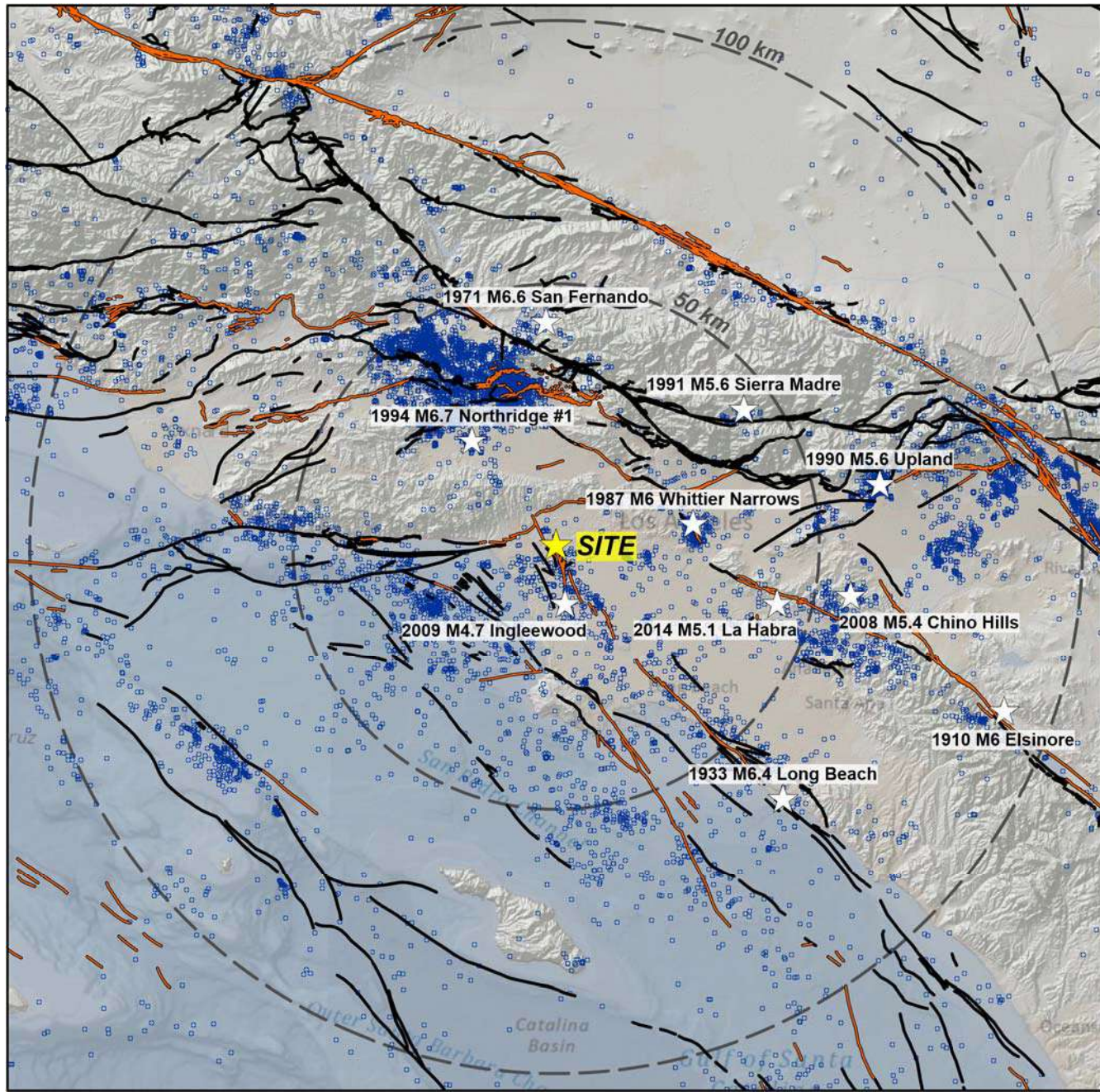
SHEAR WAVE VELOCITY PROFILE SUMMARY

Project No.: 21086A

Project: 1056 LA CIENEGA BLVD.

Date: DEC 2021



Figure F-2





Legend

USGS Quaternary Fault & Fold Database ⁽¹⁾

Age of Most Recent Displacement

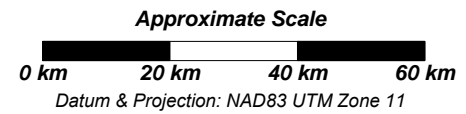
-  < 15,000 years
-  < 1,600,000 years

Seismicity ⁽²⁾

-  M ≥ 2.0
-  Historic Earthquake

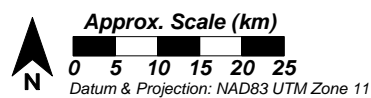
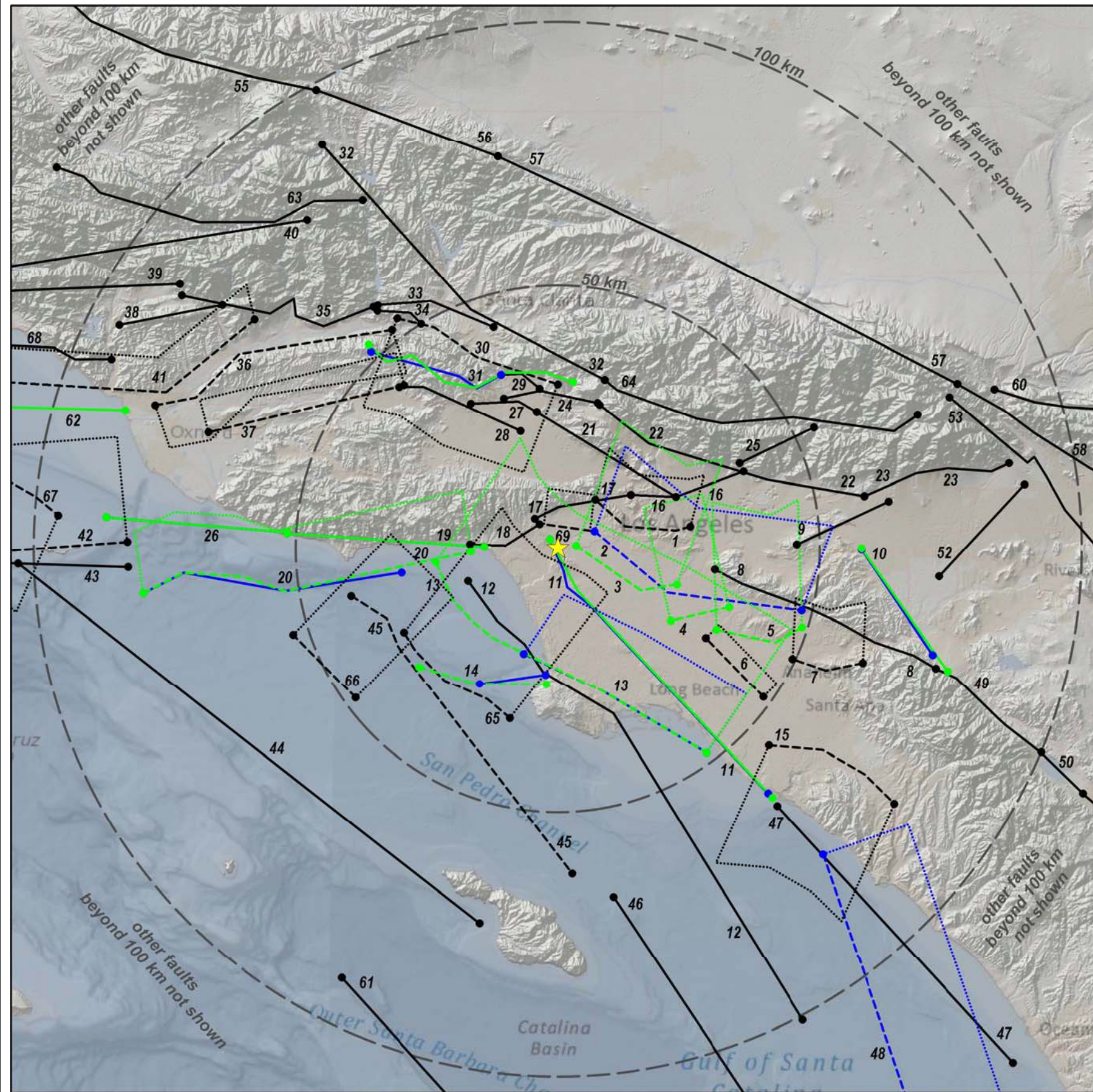
Notes:

1. Fault traces are from USGS Quaternary Fault and Fold Database (USGS, 2018).
2. Seismicity (hollow blue dots) is from Hauksson et al. (2018) catalog ("HYS" catalog). Catalog includes all instrumentally-recorded events in southern California from 01/01/1981 through 06/30/2018. Only M ≥ 2.0 events are shown here. Significant post-1900 earthquakes identified by name (white stars) are from the Southern California Earthquake Center (SCEC) online database.



REGIONAL FAULT & SEISMICITY MAP

Date: DEC 2021	Project No.: 21086A	Project: 1056 LA CIENEGA BLVD.	Figure F-3a
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No.	Fault Name
1	Elysian Park (Upper)
2	Puente Hills
3	Puente Hills (LA)
4	Puente Hills (Santa Fe Springs)
5	Puente Hills (Coyote Hills)
6	Anaheim
7	Peralta Hills
8	Elsinore - Whittier
9	San Jose
10	Chino
11	Newport-Inglewood
12	Palos Verdes
13	Compton
14	Redondo Canyon
15	San Joaquin Hills
16	Raymond
17	Hollywood
18	Santa Monica
19	Malibu Coast
20	Anacapa-Dume
21	Verdugo
22	Sierra Madre
23	Cucamonga
24	Sierra Madre (San Fernando)
25	Clamshell-Sawpit
26	Malibu Coast (Extension)
27	Mission Hills
28	Northridge Hills
29	Santa Susana East (connector)
30	Northridge
31	Santa Susana
32	San Gabriel
33	Holser
34	Del Valle
35	San Cayetano
36	Oak Ridge (Onshore)
37	Simi-Santa Rosa
38	Sisar
39	Mission Ridge-Arroyo Parida-Santa Ana
40	Santa Ynez (East)
41	Ventura-Pitas Point
42	Channel Islands Thrust
43	Santa Cruz Island
44	Santa Cruz-Catalina Ridge
45	San Pedro Basin
46	San Diego Trough North
47	Newport-Inglewood Offshore
48	Oceanside Blind Thrust
49	Elsinore - Glen Ivy
50	Elsinore - Temecula/Glen Ivy Stepmover
51	Elsinore - Temecula
52	Fontana
53	San Jacinto - San Bernardino Valley
54	San Jacinto - San Jacinto Valley
55	San Andreas - Big Bend
56	San Andreas - North Mojave
57	San Andreas - South Mojave
58	San Andreas - North San Bernardino
59	San Andreas - South San Bernardino
60	Cleghorn
61	San Clemente
62	Oak Ridge (Offshore)
63	Pine Mtn
64	San Gabriel Extension
65	San Pedro Escarpment
66	Santa Monica Bay
67	Channel Islands Western Deep Ramp
68	Red Mountain
69	San Vicente

Legend

- Surface Trace, Top of Fault (Both Fault Models)
- Blind Trace, Top of Fault (Both Fault Models)
- Blind Thrust Footprint (Both Fault Models)
- Surface Trace, Top of Fault (Fault Model 1)
- Blind Trace, Top of Fault (Fault Model 1)
- Blind Thrust Footprint (Fault Model 1)
- Surface Trace, Top of Fault (Fault Model 2)
- Blind Trace, Top of Fault (Fault Model 2)
- Blind Thrust Footprint (Fault Model 2)
- Site

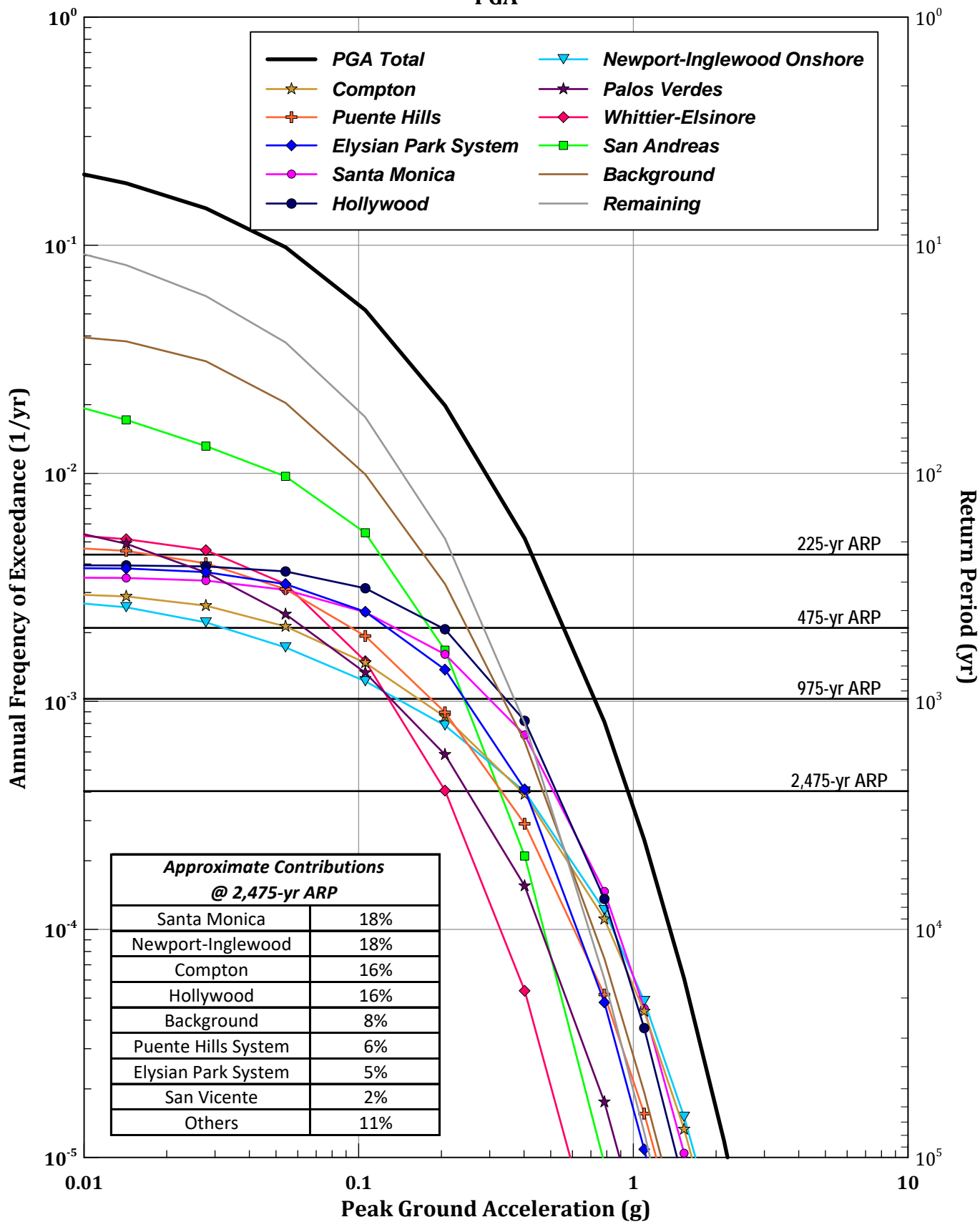
Notes:

1. All fault traces based on UCERF3 (WGCEP, 2013a) except for "Type A" faults (San Andreas, San Jacinto, Elsinore); Type A faults based on UCERF2 (WGCEP, 2008; USGS, 2009). Fault traces shown here are simplified and as-implemented in the PSHA calculations.
2. All faults within 100 km of site with slip rates greater 0.05 mm/yr are shown, except for the following: Elysian Park (Lower), North Salt Lake, Richfield, and Yorba Linda (however, these faults are included in the PSHA). Slip rates are solution mean rates from UCERF3 (WGCEP, 2013a). Only Type A faults outside 100 km are shown.
3. Fault Models 1 & 2 based on UCERF3 (WGCEP, 2013a,b). Seismic source characterization geometries for non-Type A faults are generally as shown in WGCEP (2013a,b) and slip rates are in WGCEP (2013a). Magnitude-frequency distributions approximate the SWUS WAACY model (GeoPentech, 2015) with characteristic magnitude calculated from Shaw (2009) regression. Type A faults characterized as documented in WGCEP (2008) and 2008 NSHM (Petersen et al., 2008).

SIMPLIFIED FAULT MAP FOR PSHA

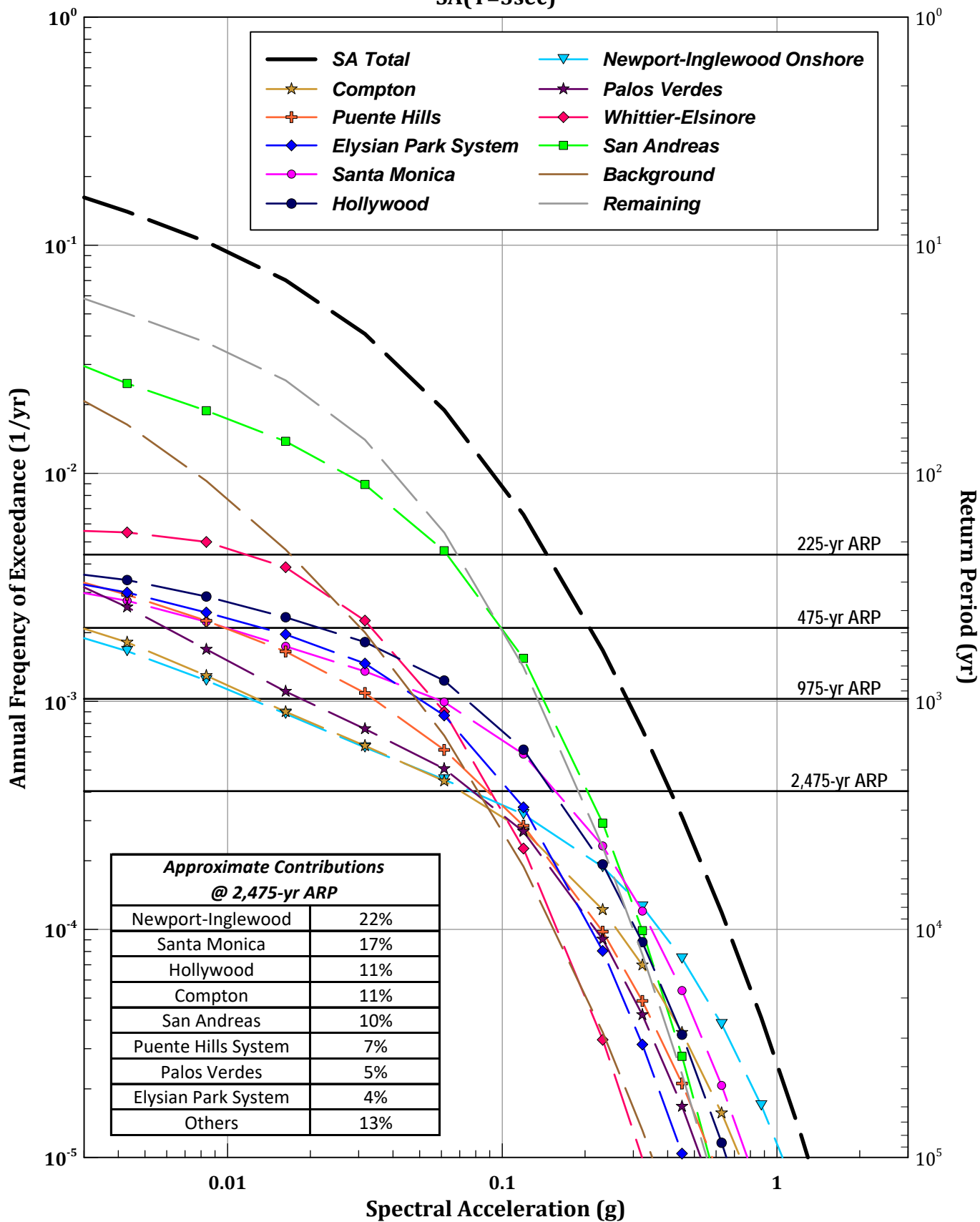
Project: 1056 LA CIENEGA BLVD.	Figure F-3b
Project No.: 21086A	Date: DEC 2021

PGA



SOURCE CONTRIBUTIONS TO THE TOTAL HAZARD AT PGA

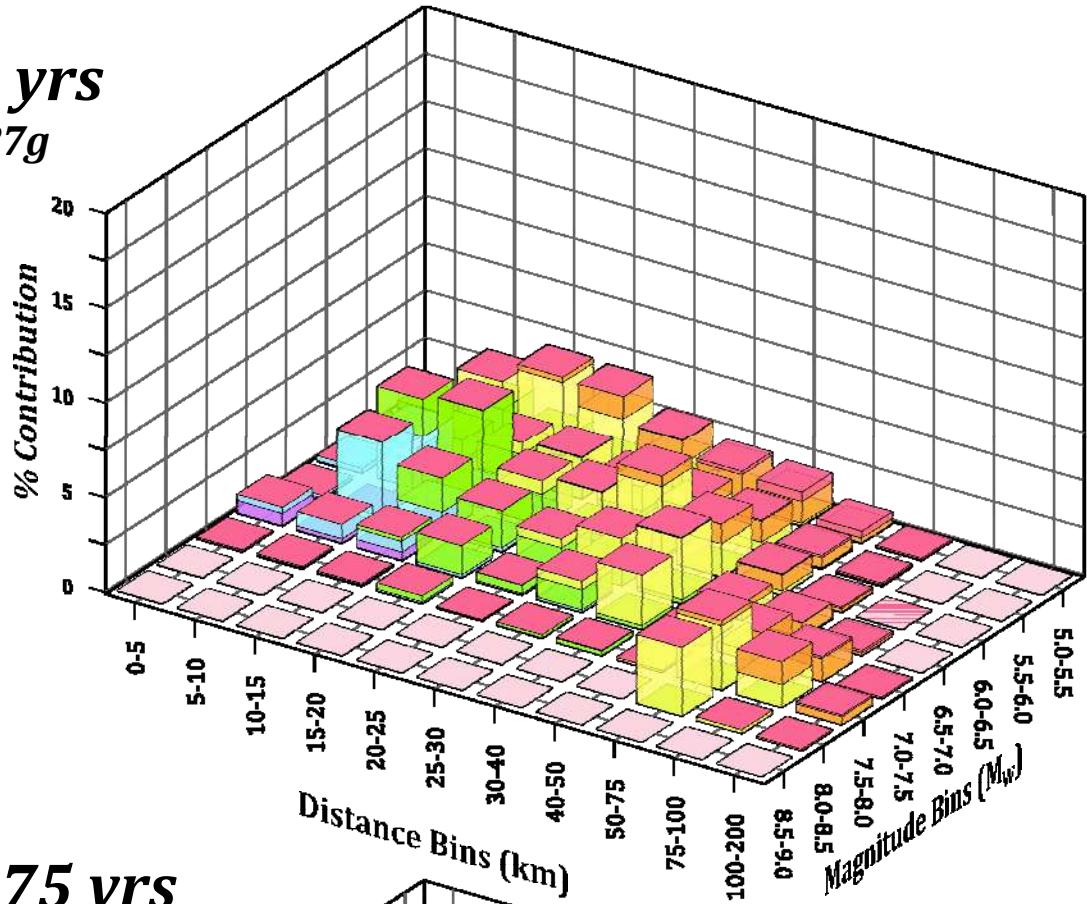
SA(T=3sec)



SOURCE CONTRIBUTIONS TO THE TOTAL HAZARD AT 3.0-SECOND SPECTRAL PERIOD

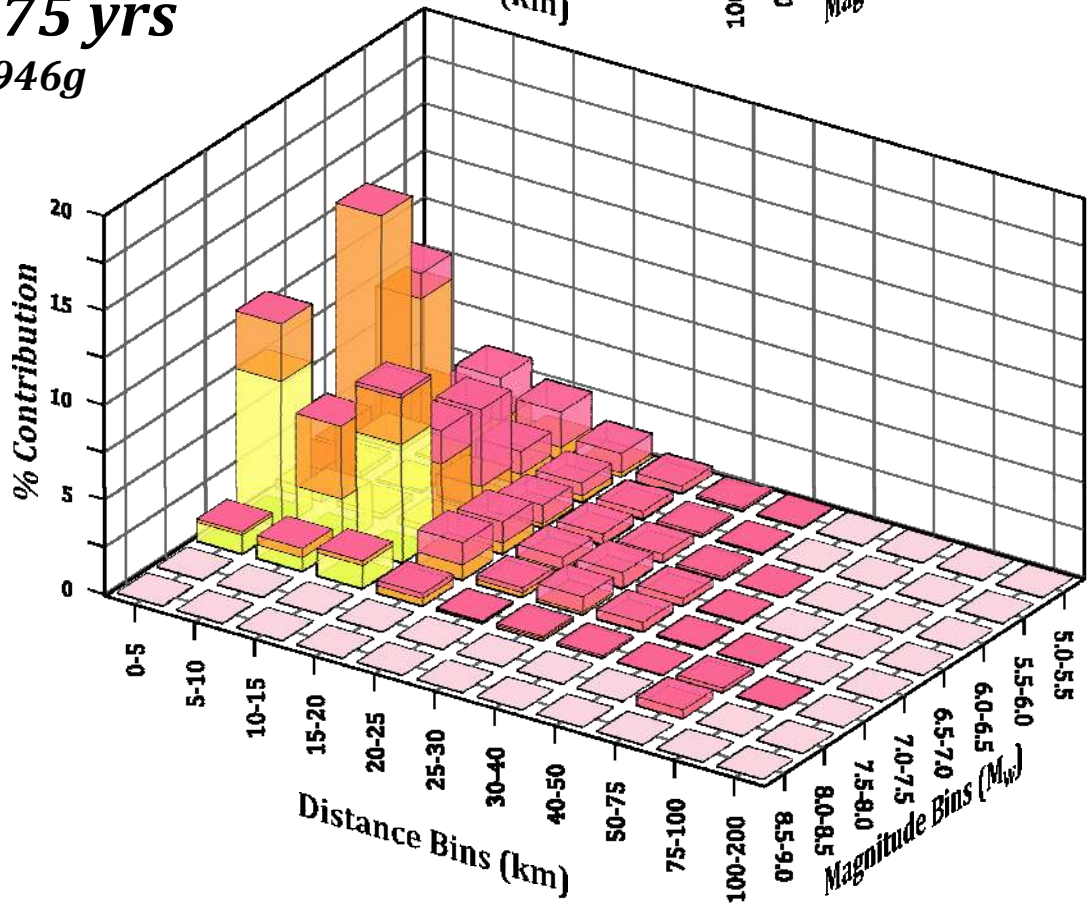
ARP = 43 yrs

PGA = 0.187g



ARP = 2475 yrs

PGA = 0.946g



Epsilon Bins

- ε: > 2
- ε: 1 to 2
- ε: 0 to 1
- ε: -1 to 0
- ε: -2 to -1
- ε: < -2

HAZARD DEAGGREGATION FOR PGA

Project No.: 21086A

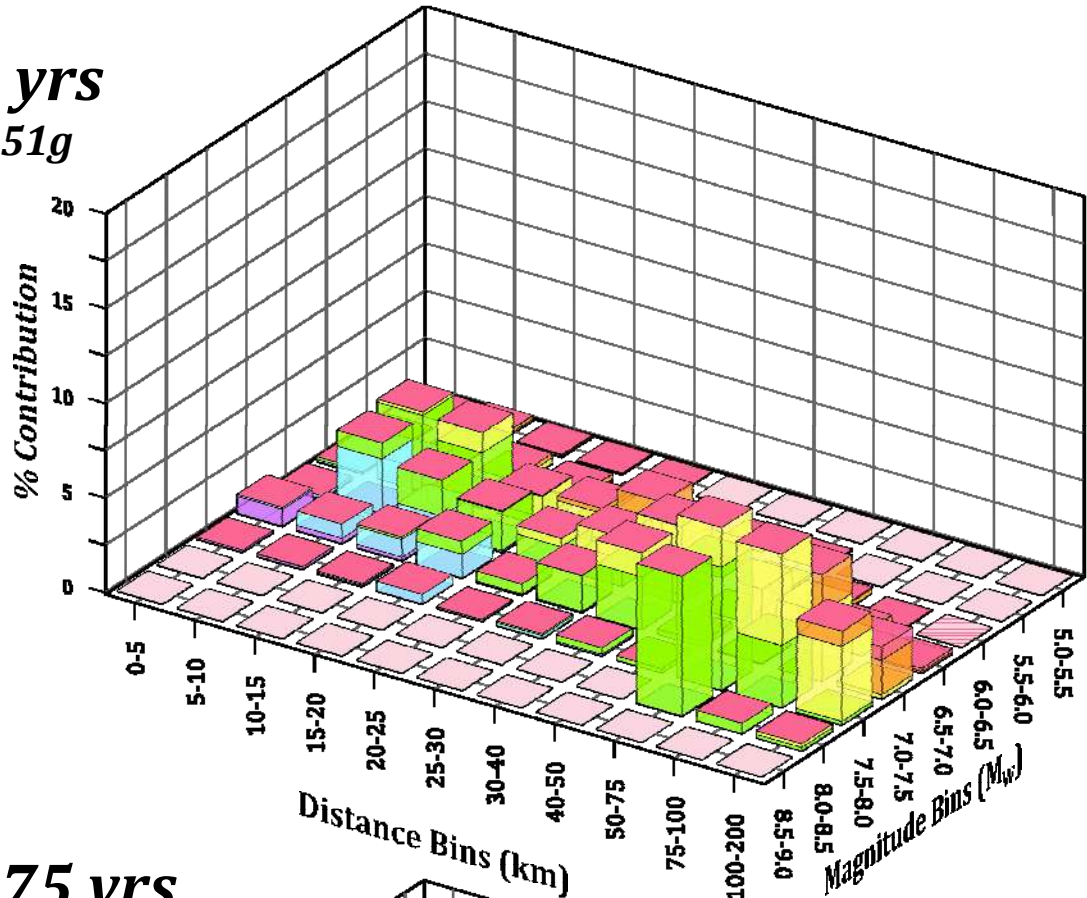
Project: 1056 LA CIENEGA BLVD.

Date: DEC 2021

Figure F-5a

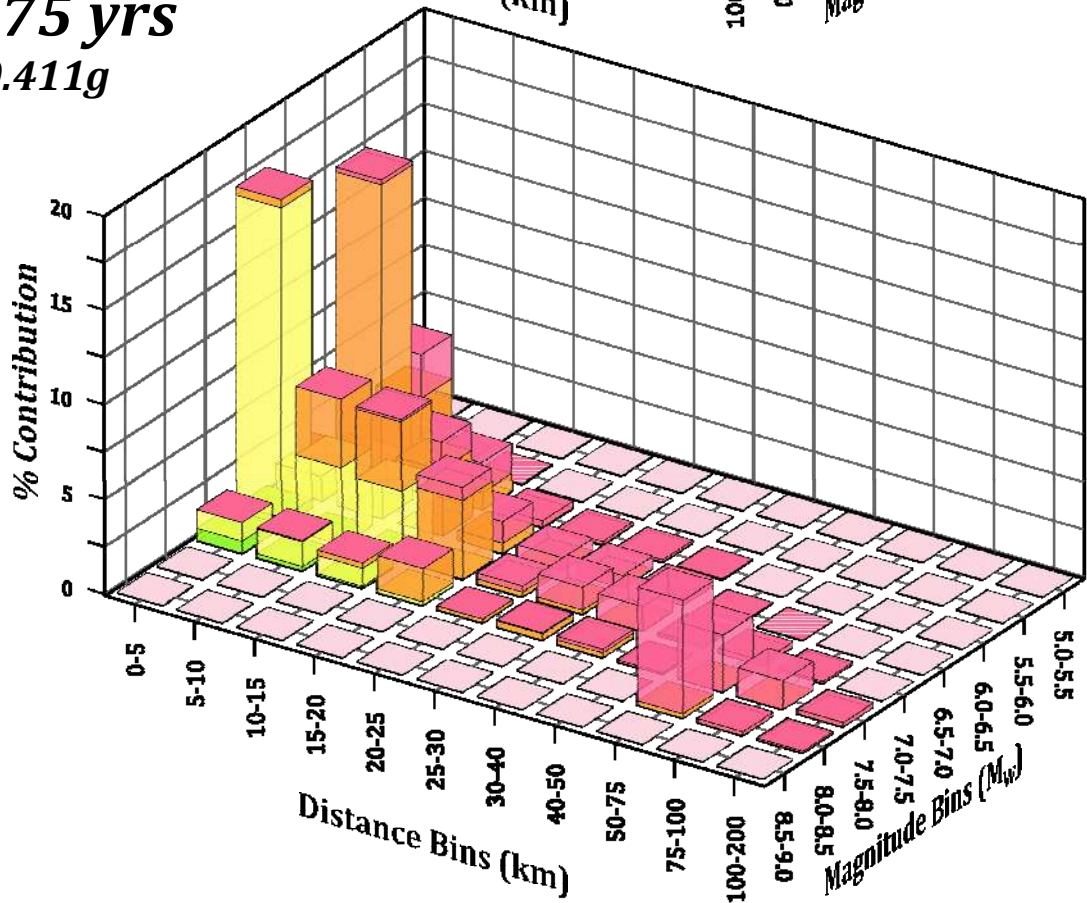
ARP = 43 yrs

SA(3s) = 0.051g



ARP = 2475 yrs

SA(3s) = 0.411g



Epsilon Bins

- ε: > 2
- ε: 1 to 2
- ε: 0 to 1
- ε: -1 to 0
- ε: -2 to -1
- ε: < -2

HAZARD DEAGGREGATION FOR 3.0-SECOND SPECTRAL PERIOD

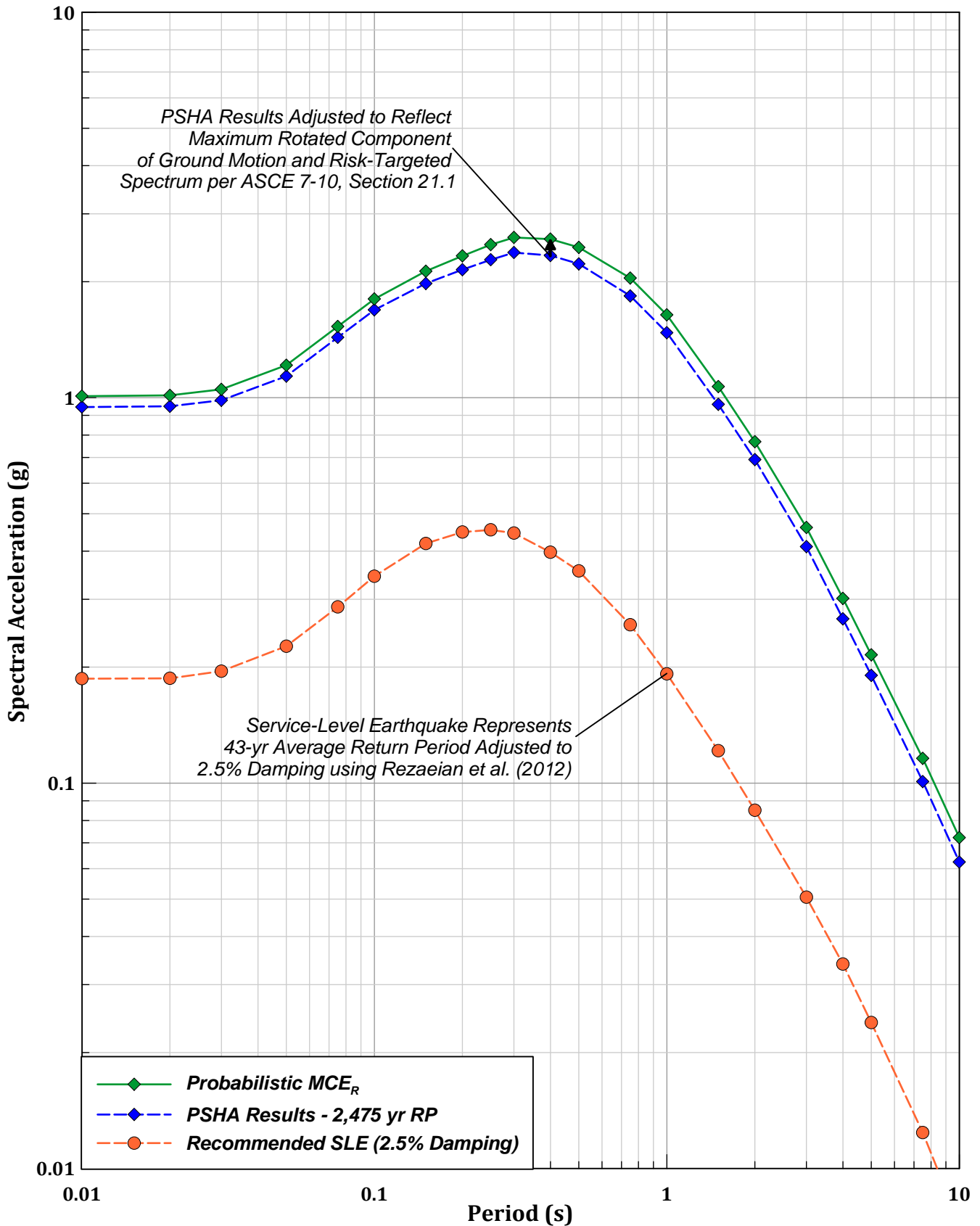
Project No.: 21086A

Project: 1056 LA CIENEGA BLVD.

Date: DEC 2021

Figure F-5b

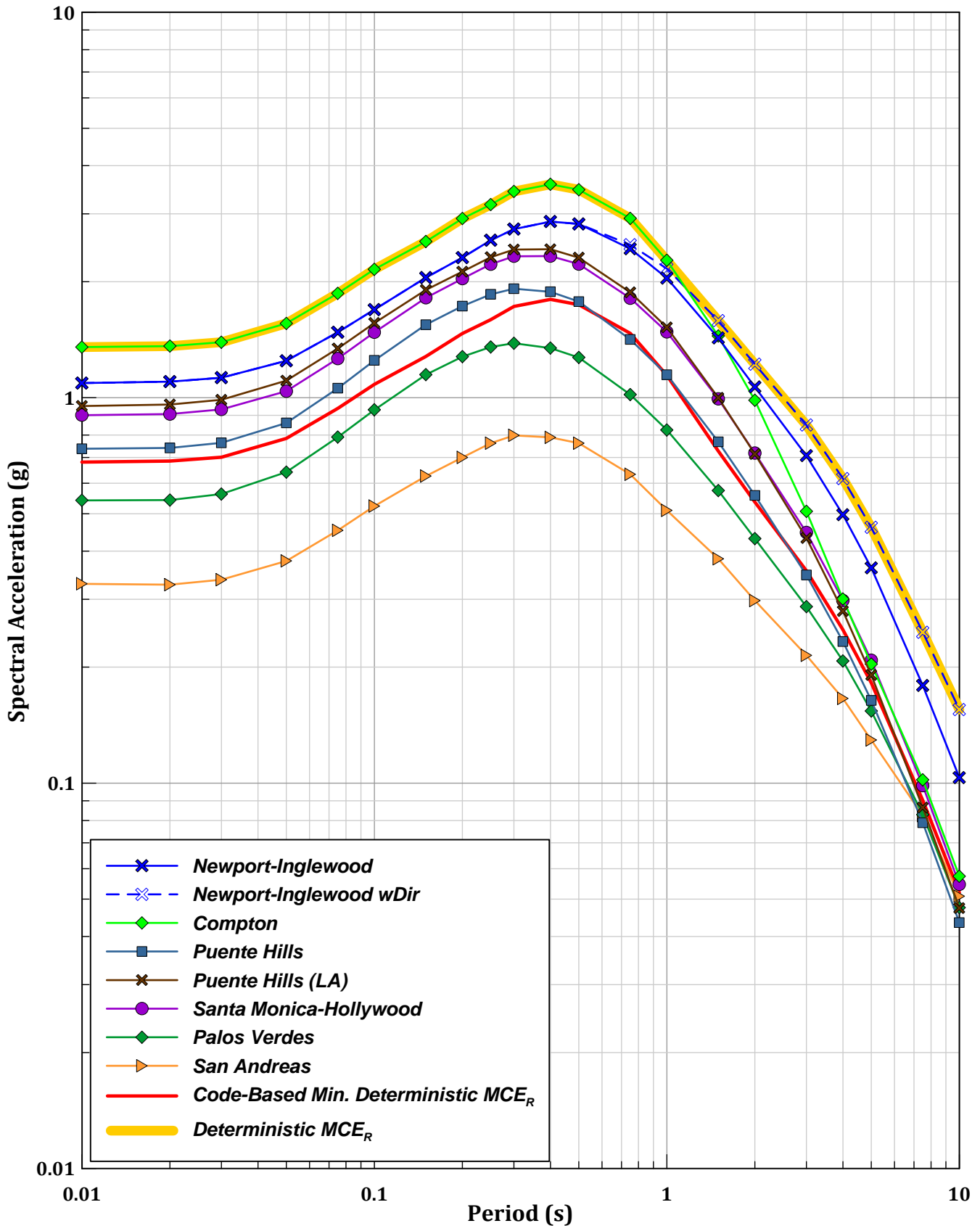
Probabilistic Spectra



Note: All spectra are for Damping (β) = 5.0% unless otherwise indicated.

PROBABILISTIC SPECTRA

Deterministic MCE Spectra



Note: All spectra are for Damping (β) = 5.0% unless otherwise indicated.

DETERMINISTIC SPECTRA

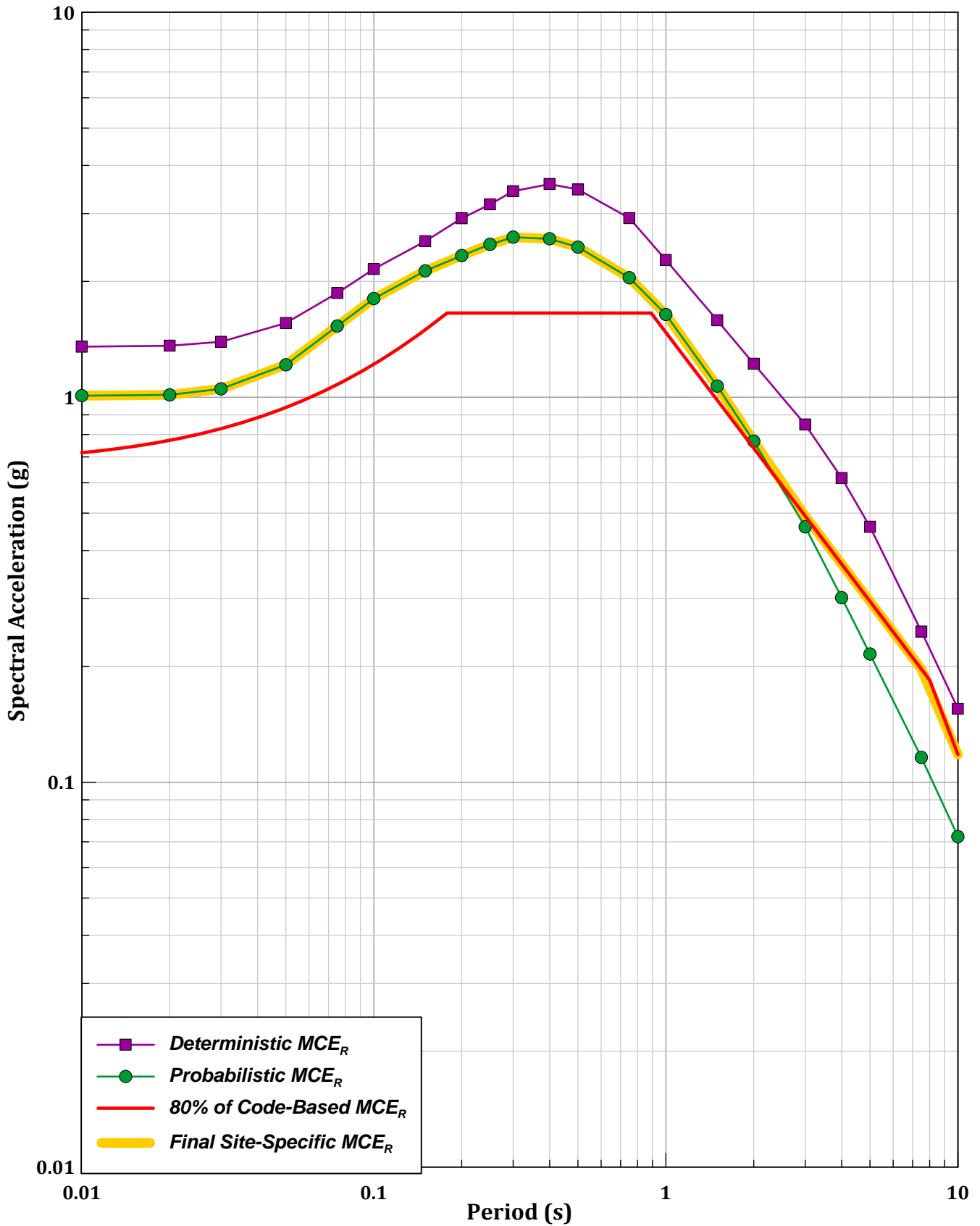
Project No.: 21086A

Project: 1056 LA CIENEGA BLVD.

Date: DEC 2021

Figure F-7

Site-Specific MCE_R Spectra



Note: All spectra are for Damping (β) = 5.0% unless otherwise indicated.

SITE-SPECIFIC MCE_R SPECTRA

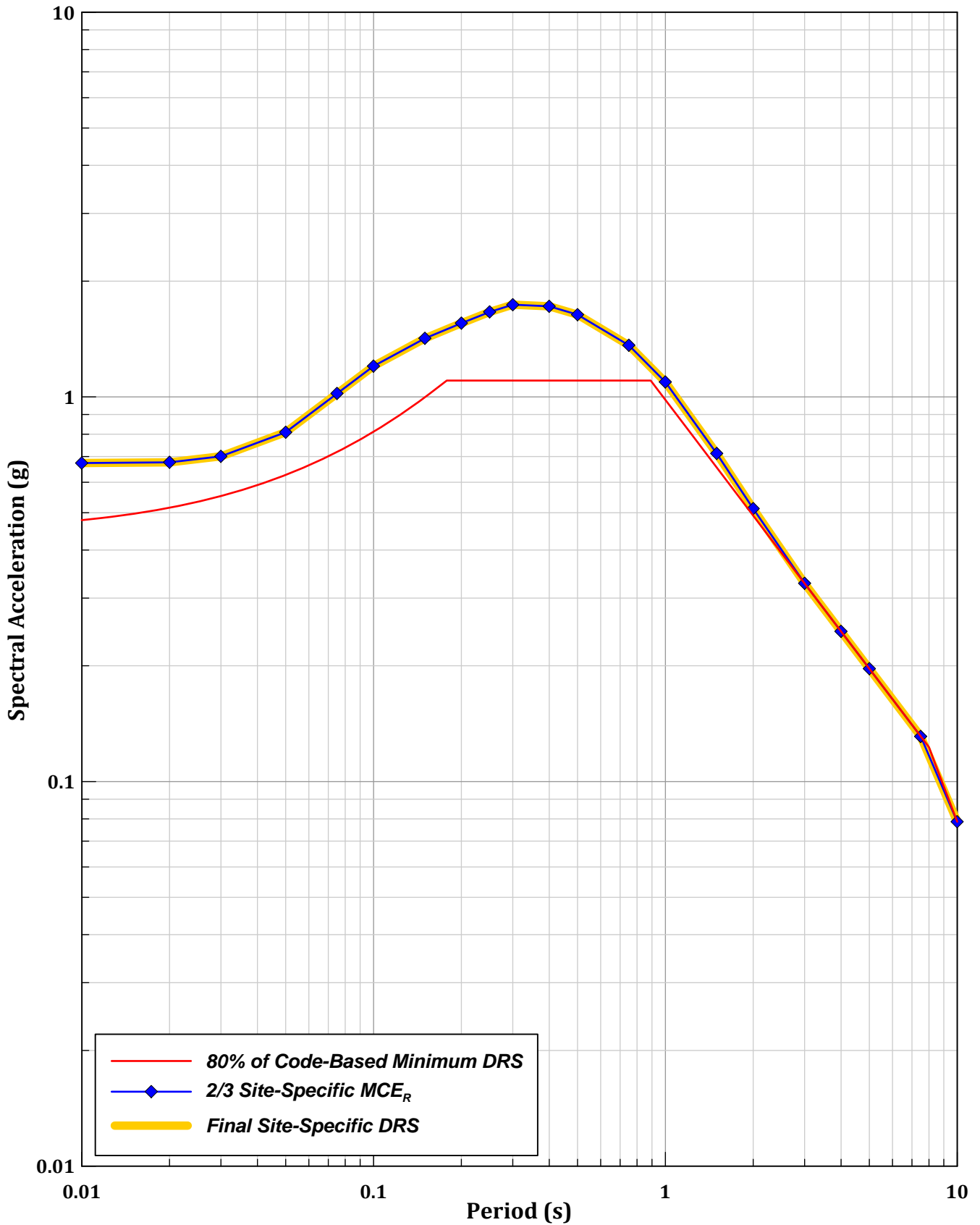
Project No.: 21086A

Project: 1056 LA CIENEGA BLVD.

Date: DEC 2021

Figure F-8

Site-Specific DRS



Note: All spectra are for Damping (β) = 5.0% unless otherwise indicated.

SITE-SPECIFIC DRS

Project No.: 21086A

Project: 1056 LA CIENEGA BLVD.

Date: DEC 2021

Figure F-9

APPENDIX G

SETTLEMENT ANALYSIS

G.1 ANALYSIS APPROACH AND RESULTS

Investigations at the site, shown in Figure G-1, encountered clay layers susceptible to consolidation settlement and sand layers susceptible to liquefaction-induced settlement. The objective of the analysis documented in this appendix is to evaluate the potential settlement under the podium and tower loading to provide an indication of the potential magnitude of settlements. For this analysis and based on the most recent communications with the Structural Engineer of Record (SEOR), the average pressures on the mat foundation beneath the Podium is considered to be 1.1 ksf applied at the bottom of a 15 ft excavation, and the Tower loading is considered to be 7 ksf applied at the bottom of a 20 ft excavation at the tower core. The sections below provide further details of the analyses.

Consolidation Settlements

For the consolidation settlement analysis, we utilized the Settle3D software package (version 4.0) by Rocscience, Inc. of Toronto, Ontario. Representative samples from clay layers throughout the site were used to perform consolidation testing the relevant soil settlement parameters were estimated from the test results based on the Casagrande and Schmertmann methods for use in the analysis.

For settlement analyses, an idealized soil profile was developed based on the subsurface sections, shown in Figures G-2a and G-2b, to represent the range of varying conditions beneath the building footprint. The material properties for the idealized soil profile, and relevant laboratory test results utilized are tabulated in Figure G-3.

Figure G-4 shows plan and isometric views of the analyzed Settle 3D model, and Figure G-5 graphically presents the model inputs and resulting settlements at various locations in the model (query points A, B, and C). Note that for simplicity, we have assumed a uniform excavation of 15 ft deep at the tower and podium locations. Further, it is noted that model properties were developed based on our field investigation and laboratory testing results presented in Appendices B, C, and D and are used in our analysis.

Note that our model calculates consolidation rebound (due to excavation) and settlements (due to structural loads) in a time-dependent manner. We have considered a 6-month gap between the end of excavation and beginning of structural loads being applied. Additionally, we've considered the design life of the project as 100 years, therefore, the calculated settlements are reported at the end of the design life of the project.

Liquefaction-Induced Settlements

The soil liquefaction potential and liquefaction-induced settlements at the site were evaluated with the methods described by Idriss & Boulanger (2014) for SPT (GP-1 through GP-4) and CPT (CPT-1

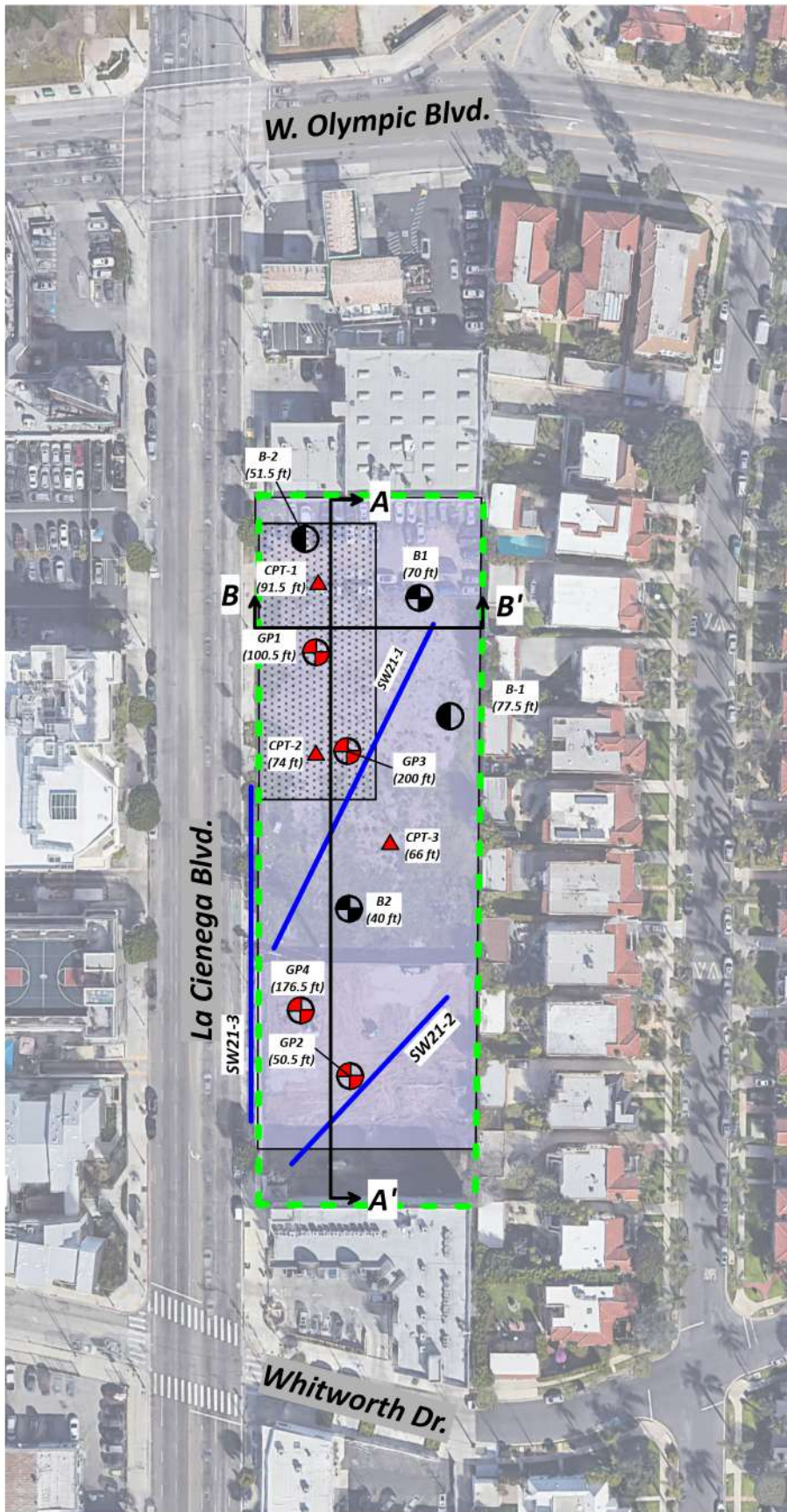
through CPT-3) measurements within the building site limits. The design input ground shaking consisted of $PGA = 0.980g$ and $M_w = 6.68$. SPT blowcounts and CPT data (tip resistance, sleeve friction, and pore pressure) were corrected for overburden stresses and stratigraphic details. CPT tip resistance data was further corrected to account for the effects of thin, interbedded soil lenses. The normalized SPT blowcounts and CPT tip resistance, shown in Figure G-6, were used to determine the cyclic resistance to liquefaction triggering. Figure G-6 also shows the Factor of Safety against liquefaction triggering and the calculated liquefaction-induced settlements.

Both the SPT and CPT liquefaction analysis procedures identified liquefiable soils in the upper 50 feet of the subsurface profile, which is consistent with the Seismic Hazard delineation for the site location. Our extensive field investigation identified interbedded non-plastic silt layers and loose sandy layers of approximately 1 to 2 foot thickness that are contributing to the liquefaction susceptibility of this site.










It should be noted that we did not consider soils that will be removed during excavation in these liquefaction analyses. Soils below the proposed excavation depth were considered to also be below the groundwater table. No dry sand settlements were considered due to the extents of the proposed excavations.

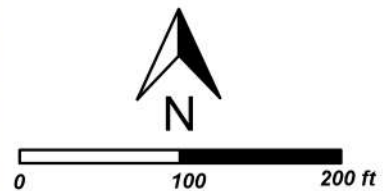
G.2 SUMMARY

Figures G-7a, G-7b, and G-7c summarize the total estimated settlements from consolidation and liquefaction analyses described above. Based on the analysis results, we estimate that for the Tower loading (i.e. average uniform 7,000 psf), the total consolidation induced settlement (total elastic and primary consolidation settlement) would be on the order of about 11 to 13 inches. For the Podium loading (i.e. uniform 1,100 psf), we estimate the total consolidation-induced settlement would be on the order of ½ to 1 inch. The results of our liquefaction evaluation indicate that the total liquefaction settlement of the saturated soils (below a design water level depth of 15 feet) are estimated to be an additional 1 to 2 inches in the event of a major earthquake.

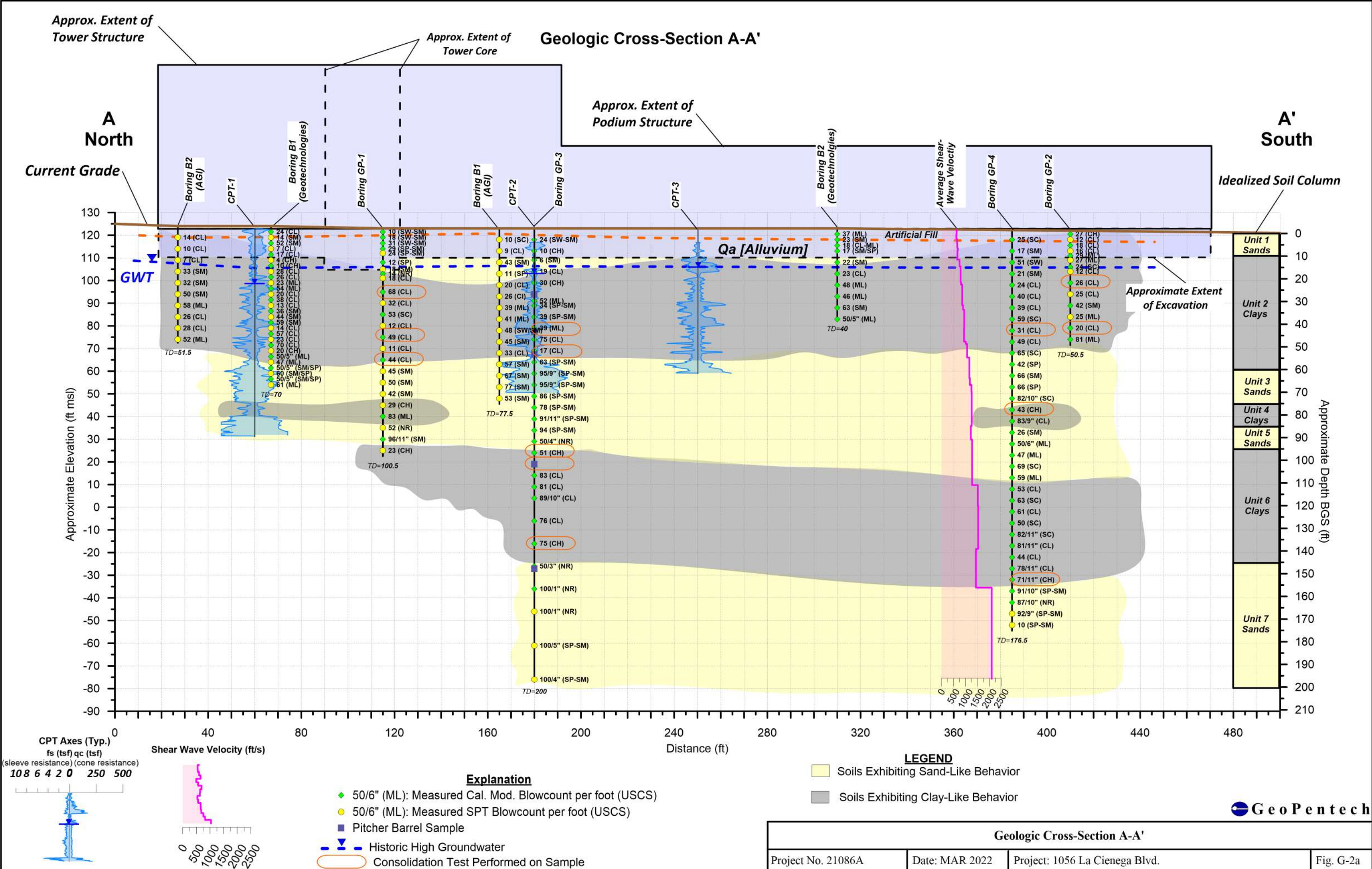


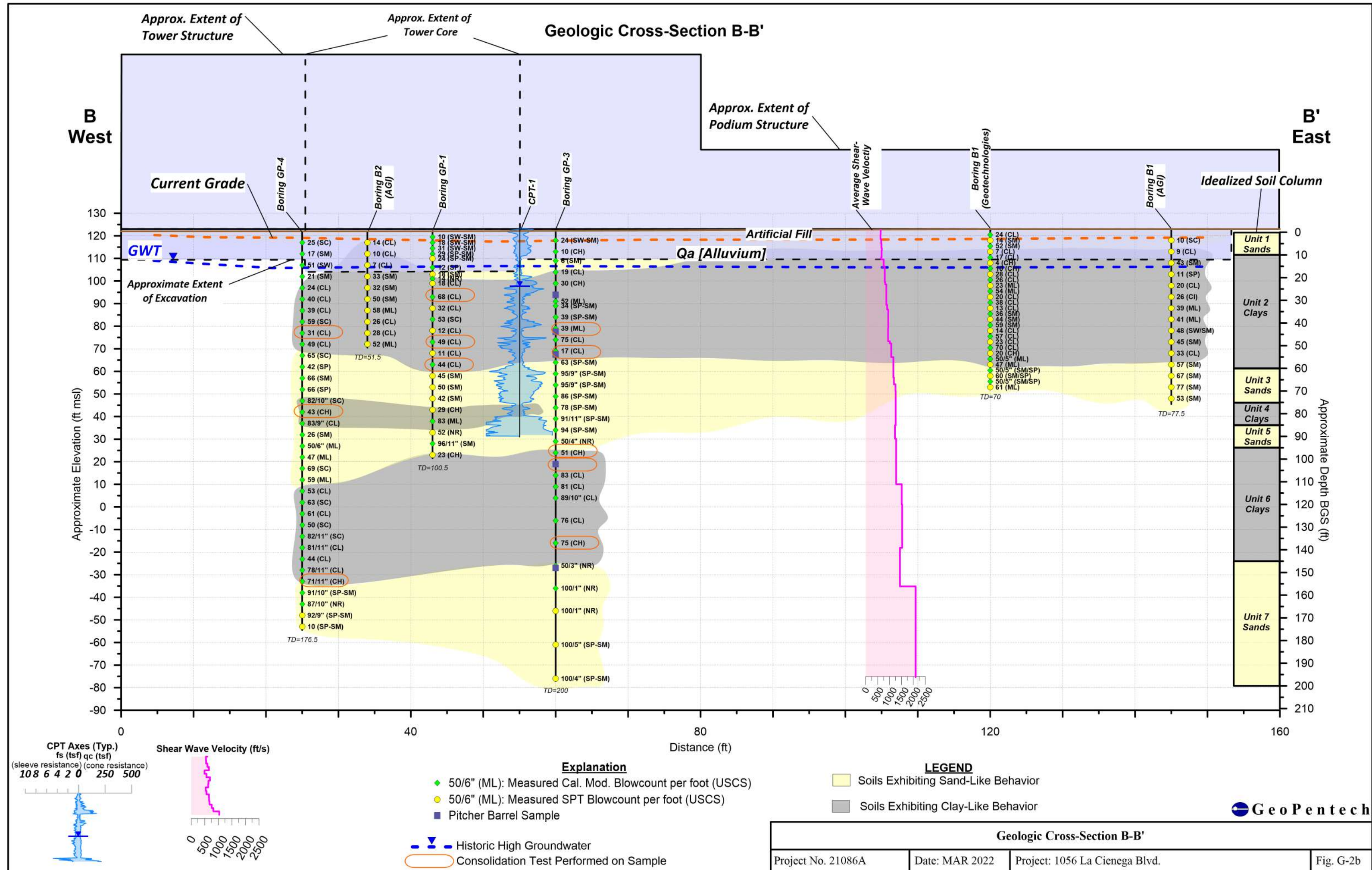
Explanations

-  Approximate Extent of Project Site
-  **B1** (50 ft) Approximate Location of Previous Borings by Geotechnologies (2012)
-  **B-1** (50 ft) Approximate Location of Previous Borings by AGI Geotechnical (2009)
-  **GP1** (50 ft) Approximate Location of Borings of Current Investigation
-  Approximate Location of Cone Penetration Tests
-  **SW21-1** MASW Lines
-  **A A'** Cross Sections
-  Approximate Extent of Tower Structure
-  Approximate Extent of Podium Structure



SITE PLAN AND BORING LOCATIONS





Interpreted Consolidation Laboratory Test Results

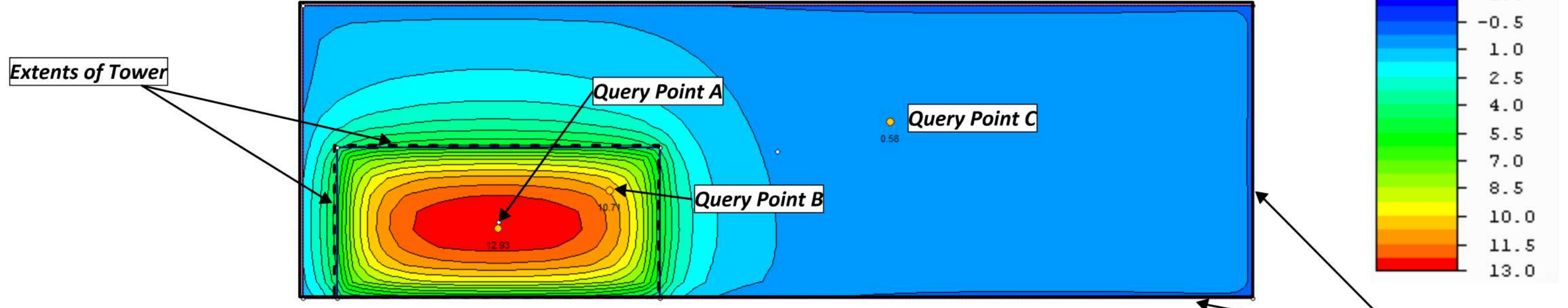
Borehole ID	Sample No.	Sample Type	Depth (ft)	USCS	C _c	C _r	σ' _p (psf)	σ' _{vo} (psf)	OCR	e ₀	C _{ce}	C _{re}	C _v Casagrande	C _v Taylor
GP-1	9a	Mod Cal	29	SC	0.069	0.011	7500	2801	2.7	0.48	0.047	0.007	-	-
GP-1	13a	Mod Cal	49	CL	0.212	0.044	9200	4064	2.3	0.77	0.120	0.025	-	-
GP-1	15b	Mod Cal	59	CL	0.180	0.033	9000	4839	1.9	0.64	0.110	0.020	-	-
GP-2	9a	Mod Cal	24	CL	0.318	0.042	6500	2341	2.8	0.87	0.170	0.022	-	-
GP-2	13a	Mod Cal	44	CL	0.322	0.046	6500	3497	1.9	0.84	0.175	0.025	-	-
GP-3	11	Pitcher Barrel	45	CL	0.118	0.023 0.018	7700	4120	1.9	0.64	0.072	0.014 0.011	300, 20	85, 32
GP-3	13a	Mod Cal	54	CL	0.167	0.016	7000	4514	1.6	0.59	0.105	0.010	-	-
GP-3	14	Pitcher Barrel	55.5	CL	0.169	0.029	8000	4490	1.8	0.66	0.102	0.017	-	-
GP-3	22	Mod Cal	99	CH	0.347	0.072 0.045	11500	6609	1.7	1.01	0.173	0.036 0.022	12, 13, 28	16, 9, 21
GP-3	23	Pitcher Barrel	104.5	CH	0.230	0.057	11000	6900	1.6	0.81	0.127	0.031	-	-
GP-3	28a	Mod Cal	139	CH	0.235	0.038	16000	9298	1.7	0.76	0.134	0.022	-	-
GP-4	9	Mod Cal	45	CL	0.240	0.039 0.020	8000	3764	2.1	0.79	0.134	0.022 0.011	-	-
GP-4	16	Mod Cal	80	CH	0.185	0.048	7500	5523	1.4	0.86	0.099	0.026	-	-
GP-4	31	Mod Cal	155	CH	0.344	0.089	9250	9731	1.0	0.86	0.185	0.048	-	-

Model Input Parameters

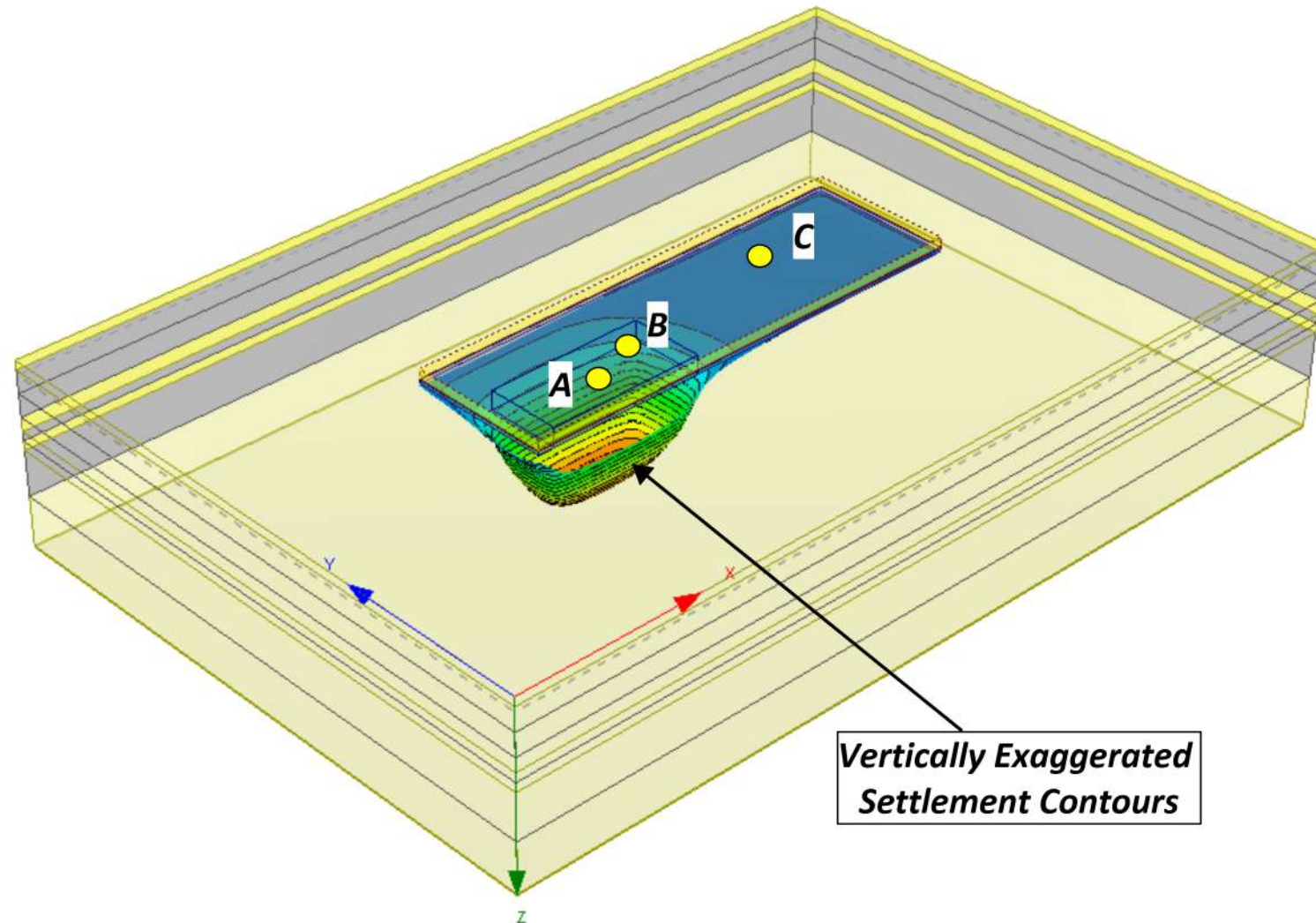
Units	Depth Range (ft)	Layer Thickness (ft)	γ _{wet} (pcf)	E (ksf)	C _{ce}	C _{re}	OCR	C _v (ft ² /yr)
Unit 1 - Sands	0-10	10	110	1000	-	-	-	-
Unit 2 - Clays	0-60	25	120	-	0.144	0.018	1.9 - 2.7	53
Unit 3 - Sands	60-75	15	125	1400	-	-	-	-
Unit 4 - Clays	75-85	10	120	-	0.144	0.018	1.9	53
Unit 5 - Sands	85-95	10	125	1600	-	-	-	-
Unit 6 - Clays	95-145	50	120	-	0.144	0.030	1.7	53
Unit 7 - Sands	145-200	55	125	2000	-	-	-	-

Model Parameter Summary	
Project: 1056 La Cienega	Figure G-3
Project No.: 21086A	Date: MAR 2022

Plan View - Settle 3D Results



Isometric View - Settle 3D Results



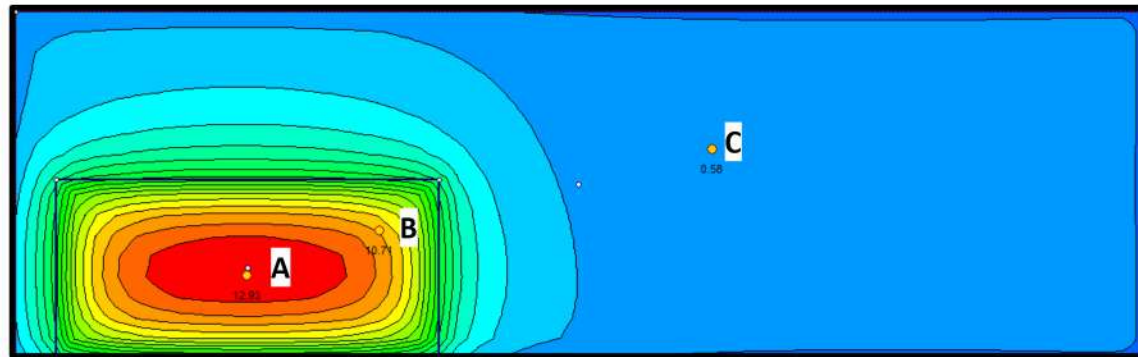
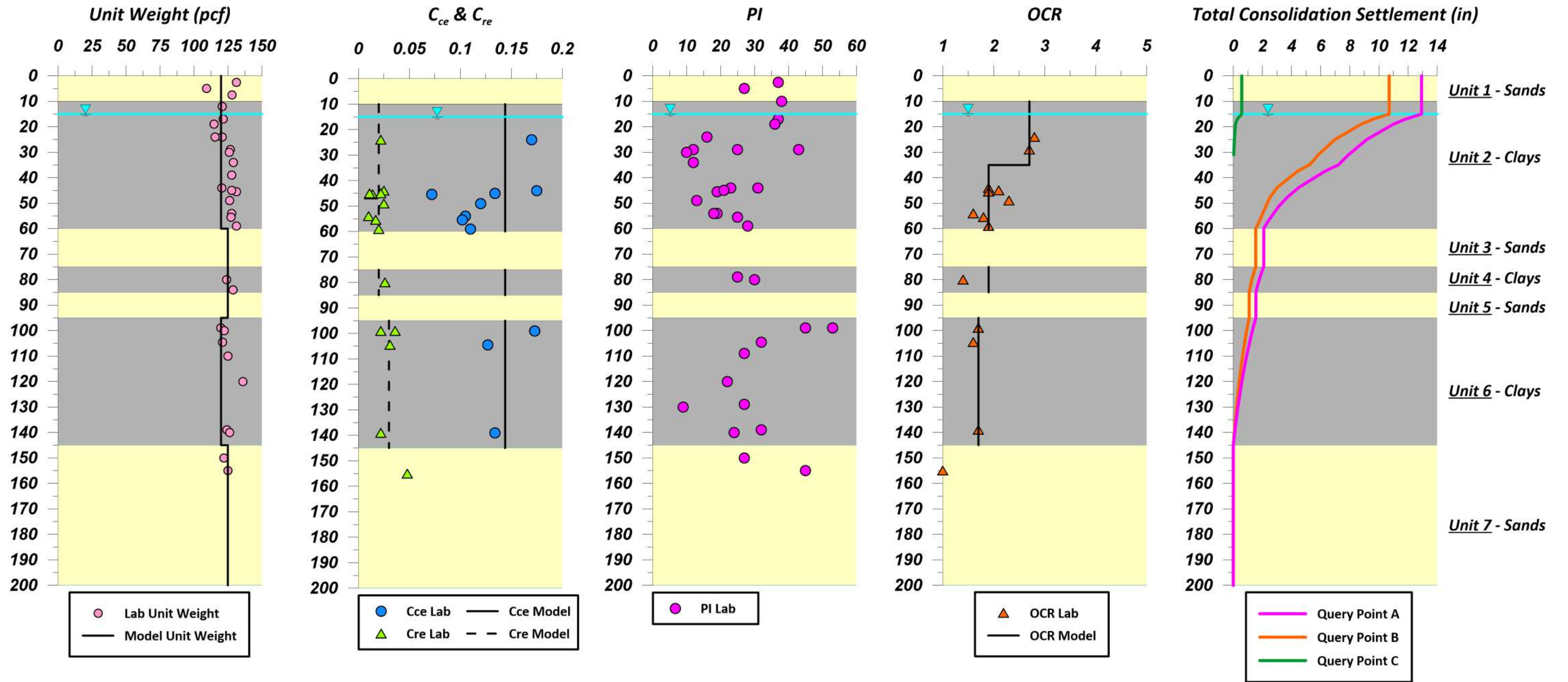
Extents of Podium

Vertically Exaggerated Settlement Contours

Note:
 Tower Loading = 7 ksf
 Podium Loading = 1.1 ksf

Settle 3D Model Results	
Project: 1056 La Cienega	Figure G-4
Project No.: 21086A	Date: MAR 2022

Representative Soil Column



Query Map

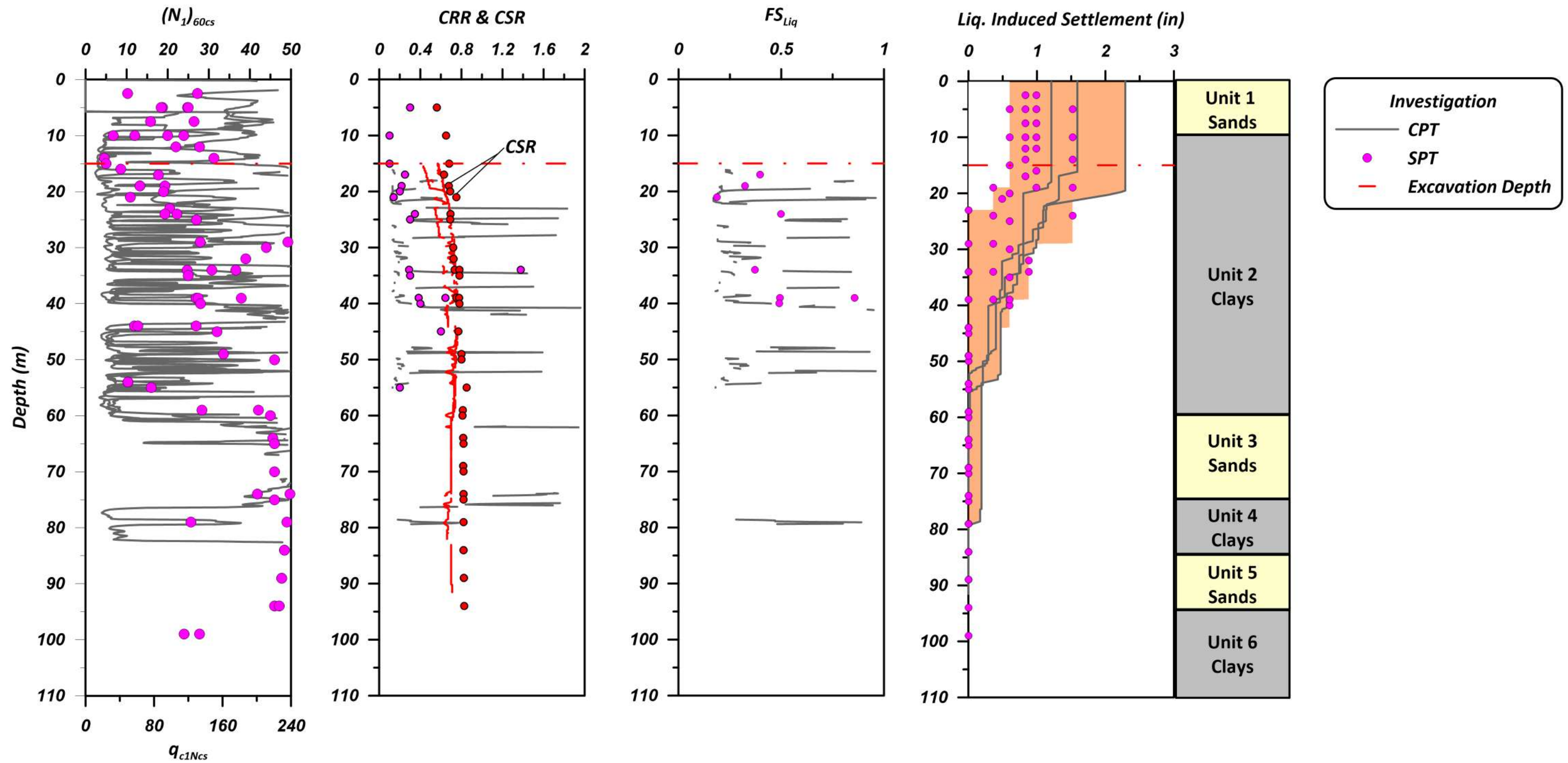
Representative Soil Column		
Project: 1056 La Cienega		Figure G-5
Project No.: 21086A	Date: MAR 2022	

Liquefaction Triggering Assessment - CPT and SPT Data

Considered Ground Motion Parameters

PGA = 0.98

M = 6.68



Note:

Only subsurface investigations performed by GeoPentech were considered in the liquefaction triggering analysis presented here. This includes borings GP-1 through GP-3 and Cone Penetration Tests CPT-1 through CPT-3.

Liquefaction Assessment - CPT & SPT Data

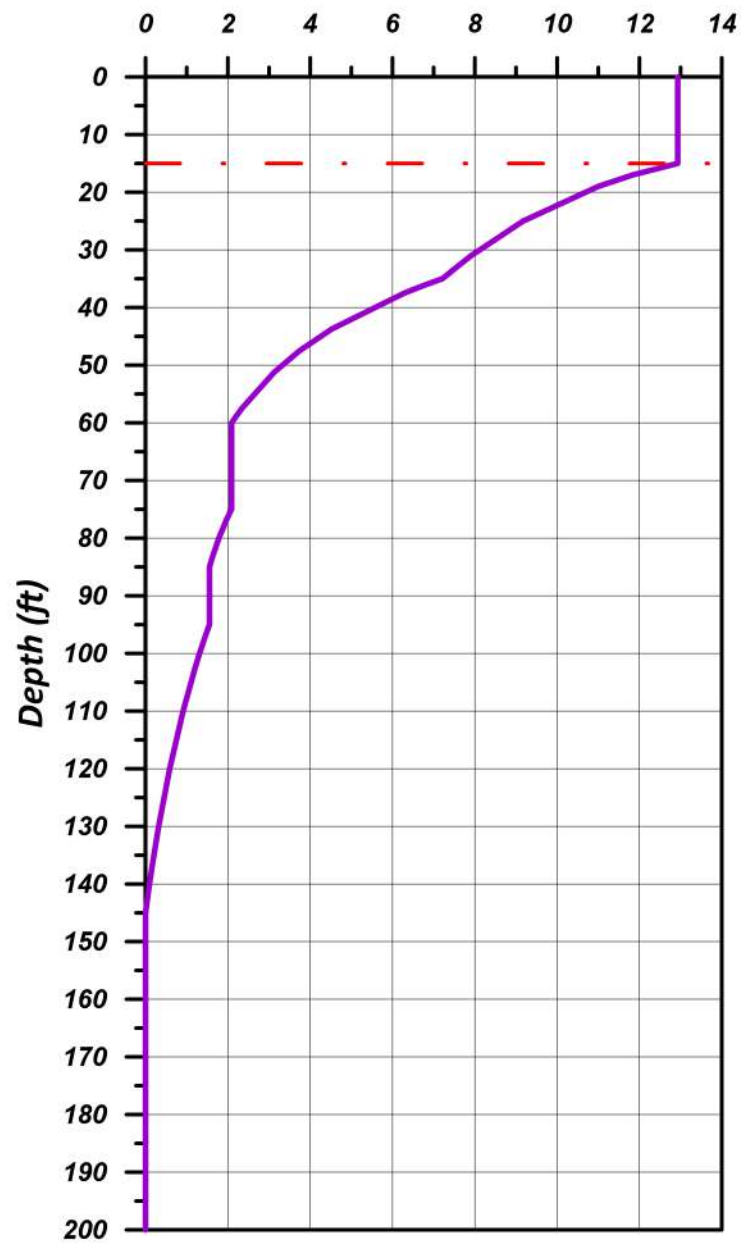
Project: 1056 La Cienega

Figure G-6

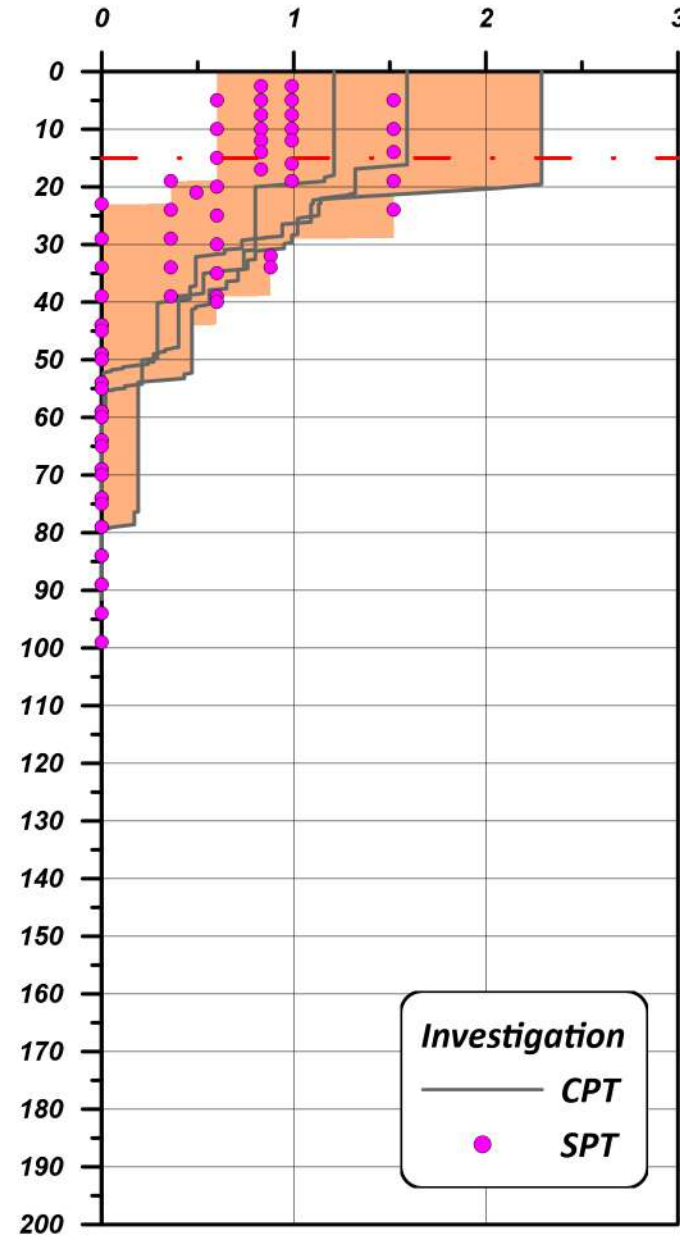
Project No.: 21086A

Date: MAR 2022

Consolidation Settlement (in)

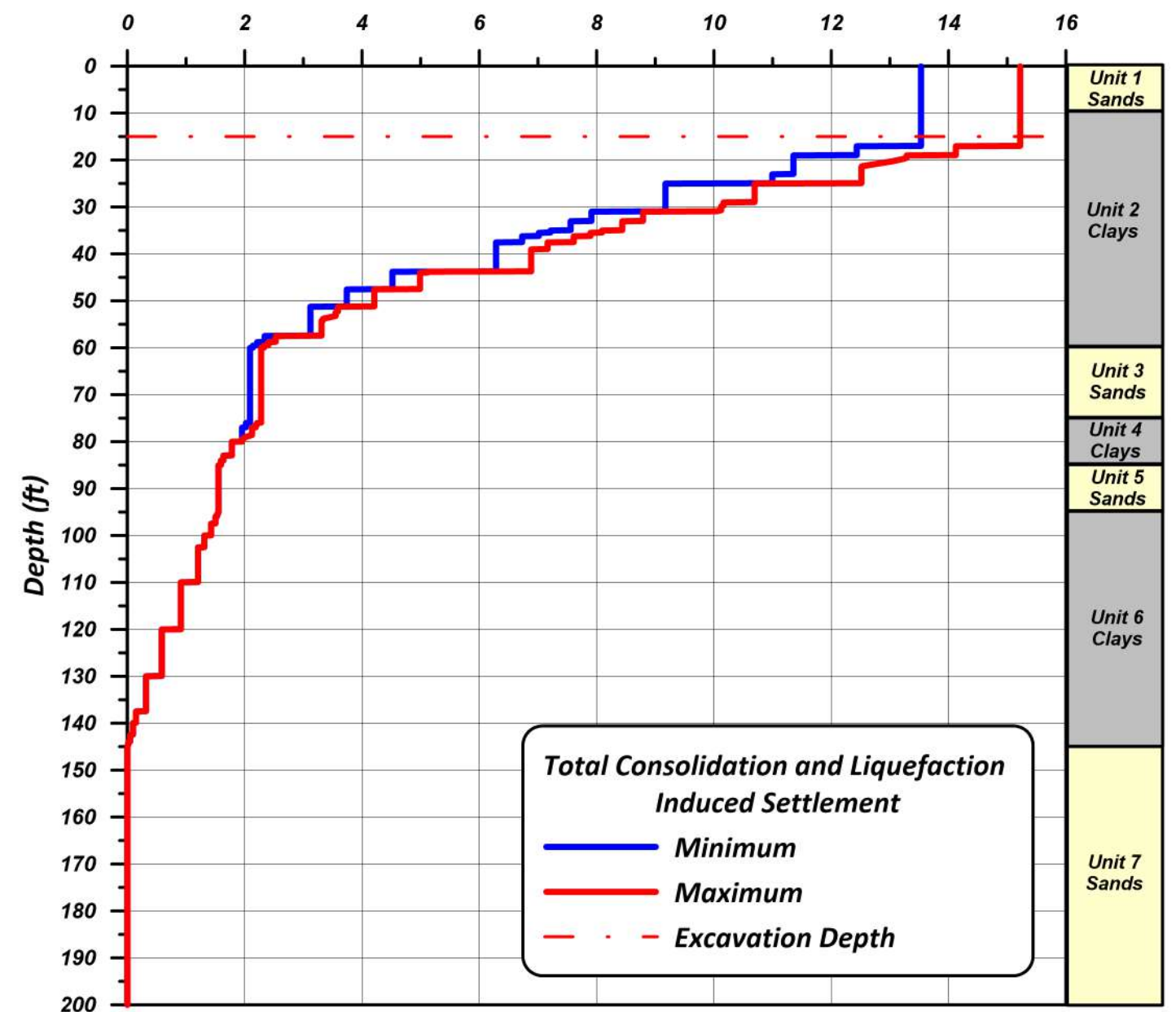


Liquefaction Induced Settlement (in)

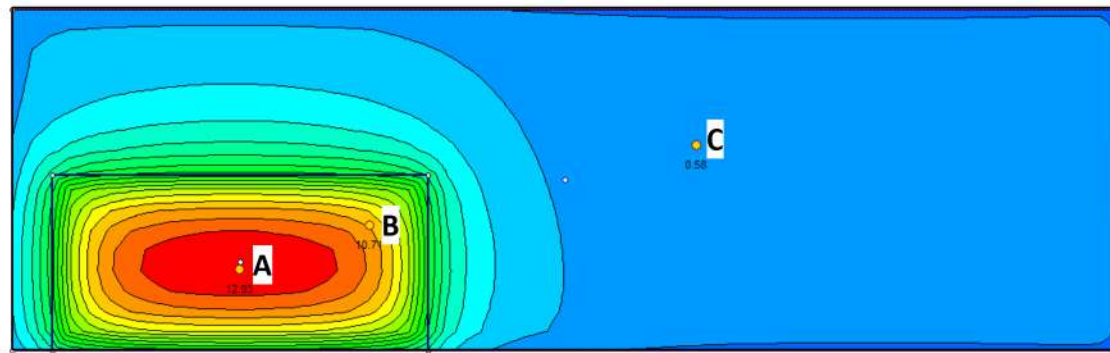


Investigation
 — CPT
 ● SPT

Total Consolidation and Liquefaction Induced Settlement (in)



Total Consolidation and Liquefaction Induced Settlement
 — Minimum
 — Maximum
 - - - Excavation Depth

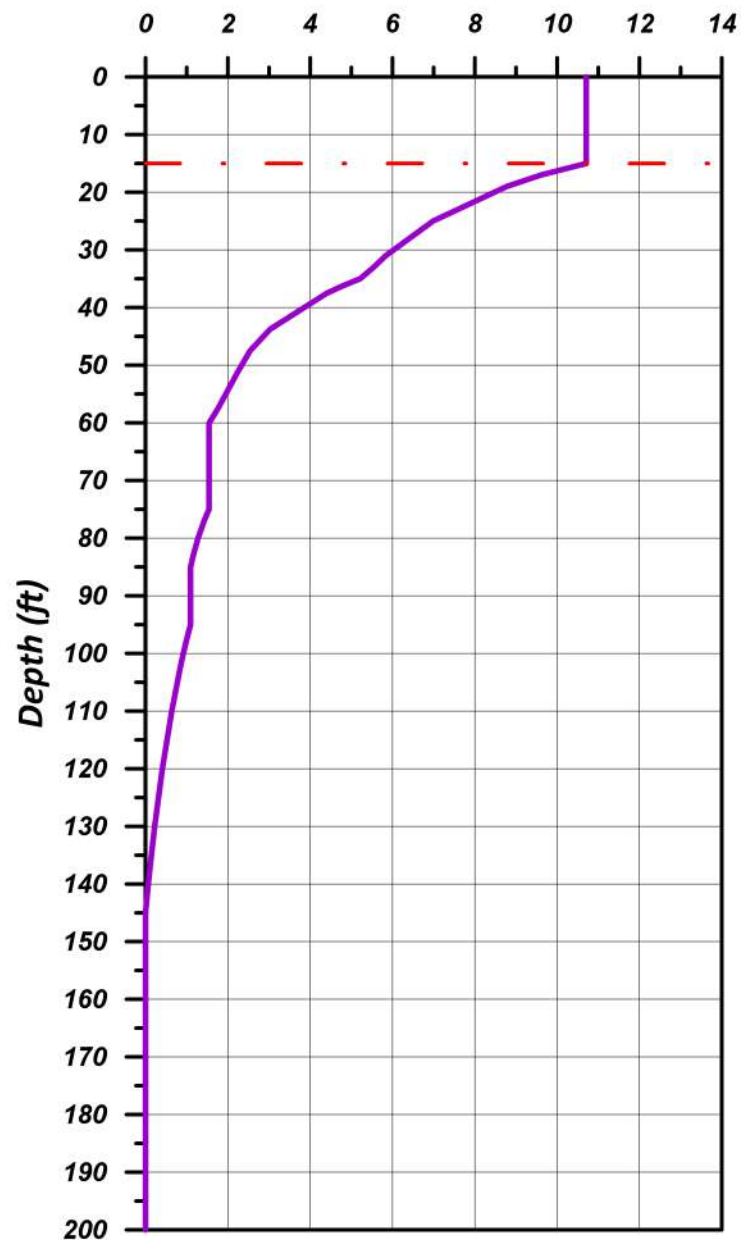


Query Map

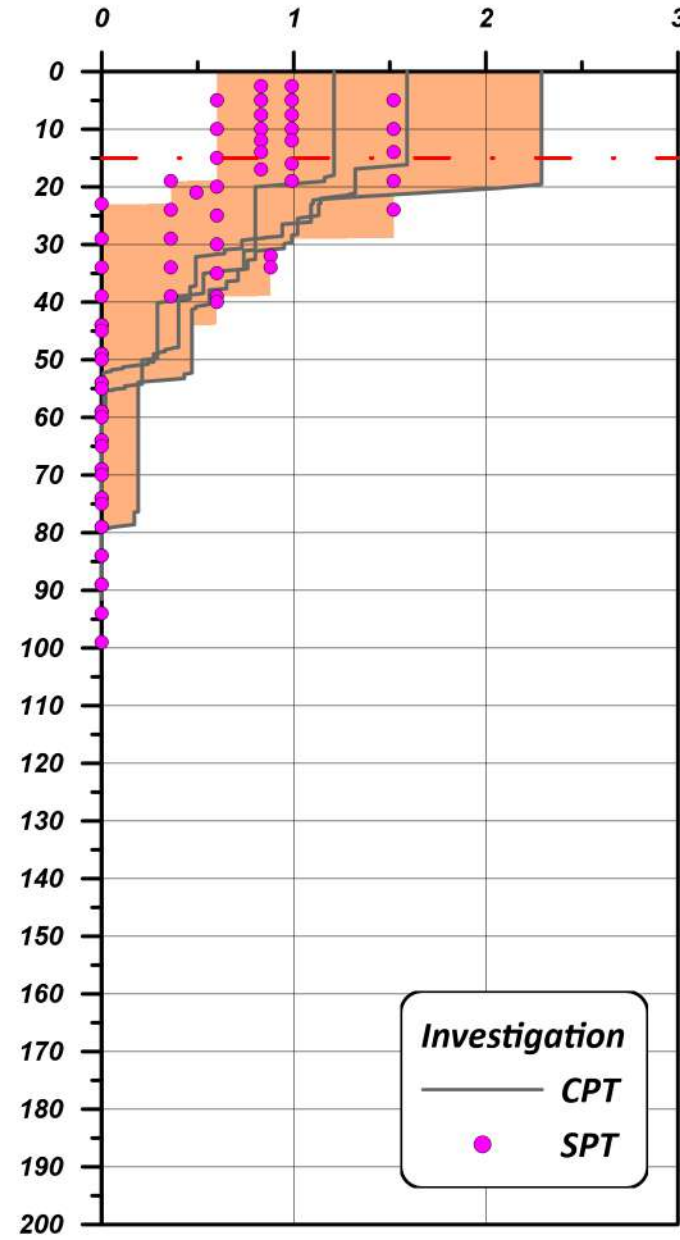
Loading: 7 ksf Tower Pressure
 1.1 ksf Podium Pressure

Imposed Settlement Summary - Query Point A	
Project: 1056 La Cienega	Figure G-7a
Project No.: 21086A	Date: MAR 2022

Consolidation Settlement (in)

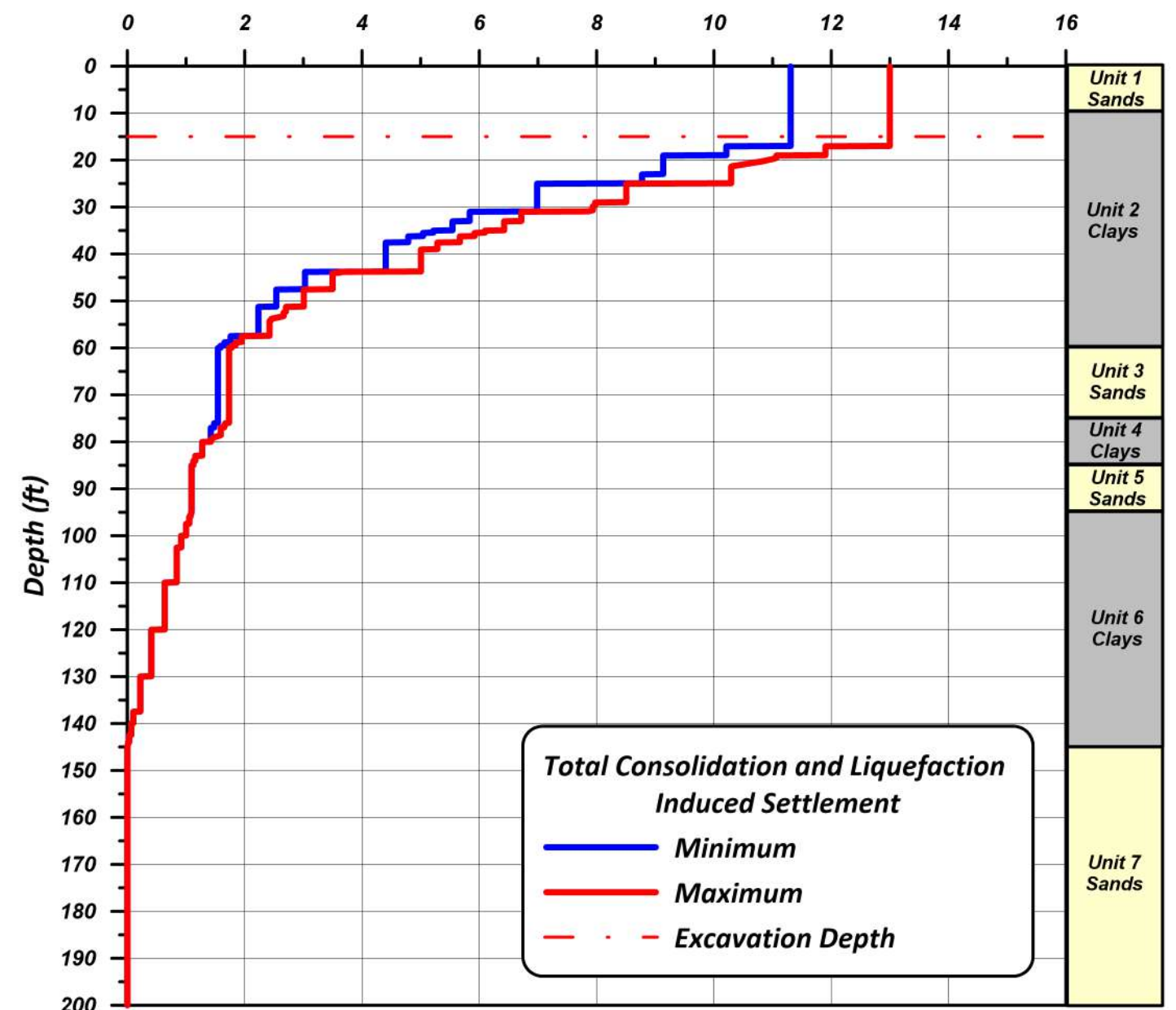


Liquefaction Induced Settlement (in)



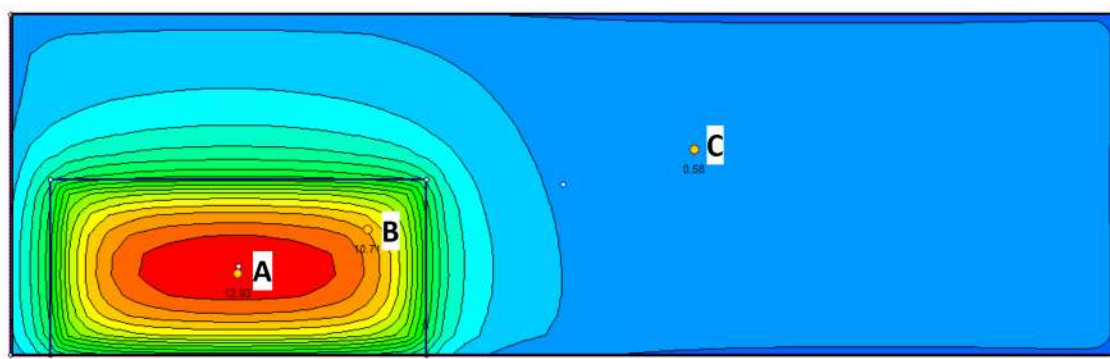
Investigation
 — CPT
 ● SPT

Total Consolidation and Liquefaction Induced Settlement (in)



Total Consolidation and Liquefaction Induced Settlement
 — Minimum
 — Maximum
 - - - Excavation Depth

- Unit 1 Sands
- Unit 2 Clays
- Unit 3 Sands
- Unit 4 Clays
- Unit 5 Sands
- Unit 6 Clays
- Unit 7 Sands

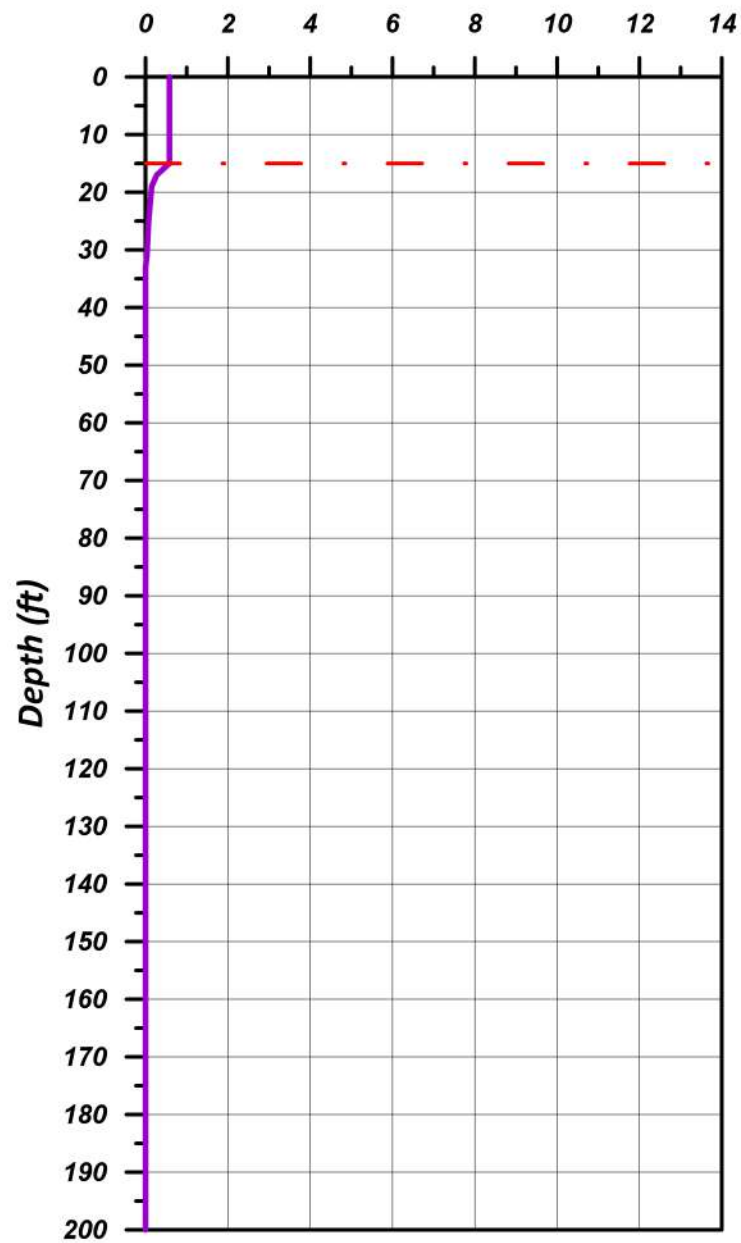


Query Map

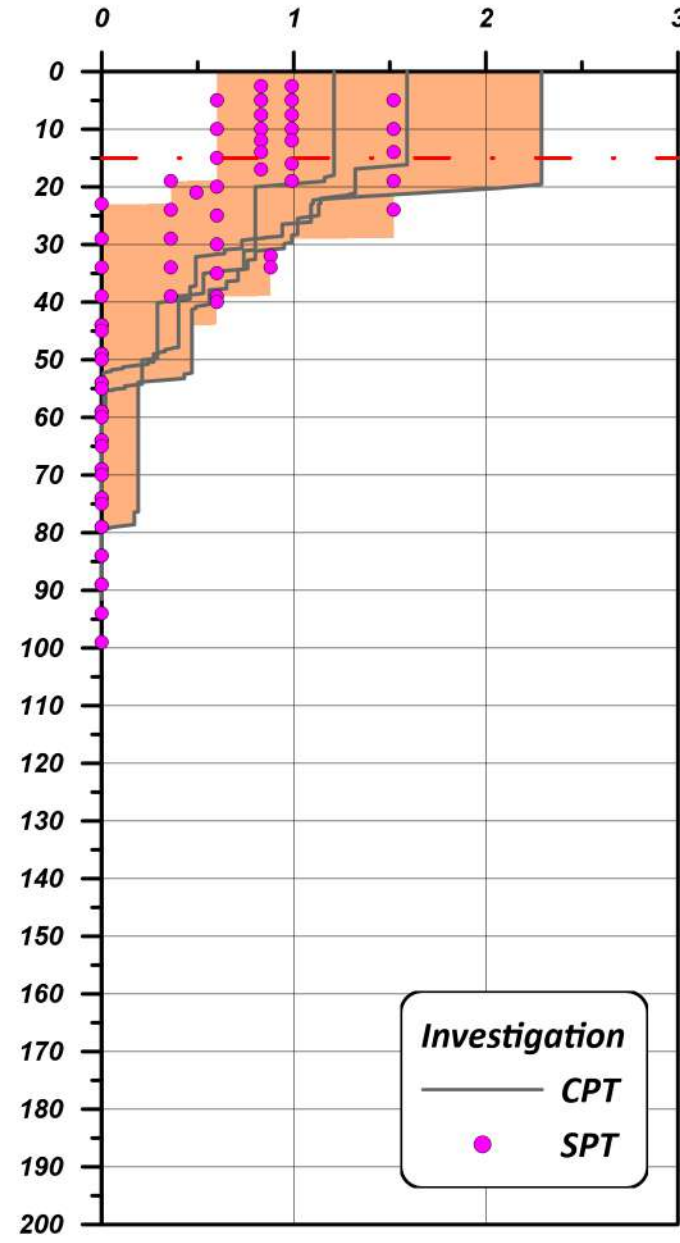
Loading: 7 ksf Tower Pressure
 1.1 ksf Podium Pressure

Imposed Settlement Summary - Query Point B	
Project: 1056 La Cienega	Figure G-7b
Project No.: 21086A	Date: MAR 2022

Consolidation Settlement (in)

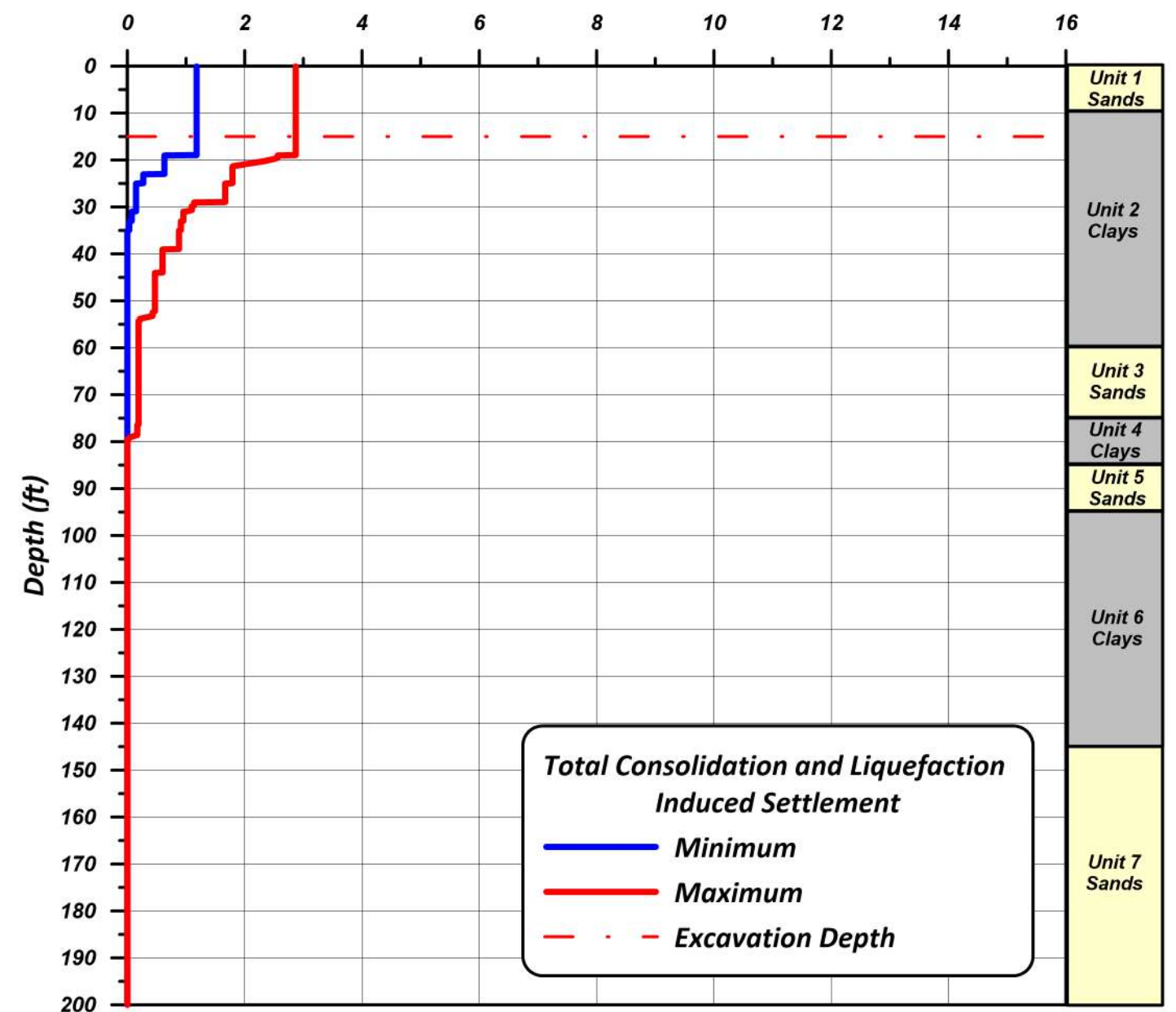


Liquefaction Induced Settlement (in)

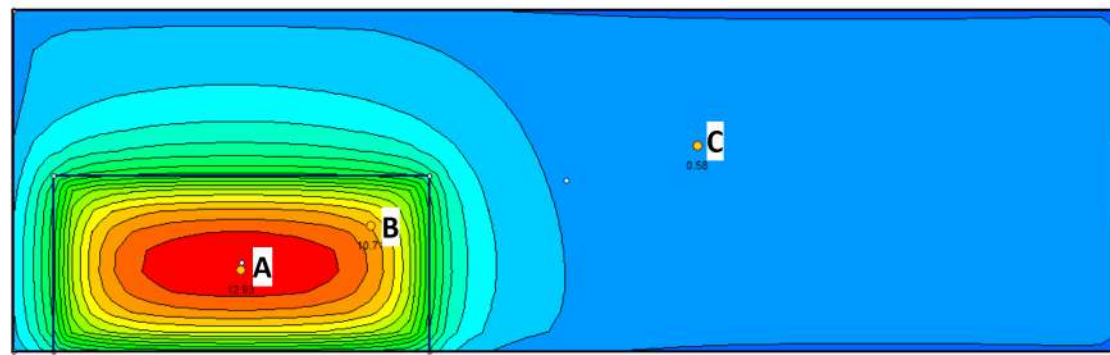


Investigation
 — CPT
 ● SPT

Total Consolidation and Liquefaction Induced Settlement (in)



Total Consolidation and Liquefaction Induced Settlement
 — Minimum
 — Maximum
 - - - Excavation Depth



Query Map

Loading: 7 ksf Tower Pressure
 1.1 ksf Podium Pressure

Imposed Settlement Summary - Query Point C	
Project: 1056 La Cienega	Figure G-7c
Project No.: 21086A	Date: MAR 2022

APPENDIX H

PILE ANALYSIS RESULTS

This appendix summarizes the assumptions and results of our analysis for axial and lateral pile capacities in the following sections.

H.1 MODEL PARAMETERS

Figure H-1 summarizes the results of direct shear strength testing performed on select soil samples and tabulates strength values in terms of cohesion and peak friction angle. Additionally, Figure H-1 shows the values that were considered for the axial and lateral pile analyses in our model.

H.2 AXIAL PILE ANALYSIS RESULTS

The estimated allowable downward and upward capacities of 36-, 42- and 48-inch-diameter drilled cast-in-place concrete piles extending to unit 7 of the subsurface soils are presented on Figure H-2. A minimum tipping elevation of -30 ft msl is recommended to ensure the piles extend below unit 6 clayey soils and into unit 7. The capacities are based on skin friction only, and end bearing has not been included.

H.3 LATERAL PILE ANALYSIS RESULTS

Moment and shear forces developed in a single drilled cast-in-place concrete pile were evaluated using LPILE V5.0 by Ensoft Inc.

LPILE computes deflection, shear, bending moment and soil response with respect to depth in nonlinear soils. Soil behavior is modeled with p-y curves (resistance-deflection) curves that can be generated internally or input by the user, depending on the nature of the soil profile. A number of pile head fixity conditions can be specified, and pile properties can be varied as a function of depth.

The soil property profile and assumed pile properties used in the analyses are presented in Figure H-1 and Table H-1, respectively. The parameters and boundary conditions used in individual LPILE runs are presented in Table H-2. Output plots from the individual runs are included in Figure H-4 through H-8. Summaries of the load-deflection response at the top of the piles for both fixity conditions are shown in Figure H-9.

Lateral Pile Analysis

Table H-1	Pile Properties				
Pile Type	Diameter, in	Pile Length Below Pile Cap, ft	Pile Area, in ¹	Elastic Modulus of Pile Cross-Section, psi ¹	Moment of Inertia of Pile Cross-Section, in ⁴
36-inch Diameter Drilled Pile	36	140	1018	3,600,000	82,448
42-inch Diameter Drilled Pile	42	140	1385	3,600,000	152,745
48-inch Diameter Drilled Pile	48	140	1810	3,600,000	260,575

1- Piles were analyzed as elastic members for purposes of developing moment and shear force information to aid in structural design of pile sections.

Table H-2	Cases Analyzed		
Run Identification No.	Pile Type	Pile Head Fixity Condition	Output Files Attached
36-inch Diameter Drilled Pile	Drilled Cast-in-Place Concrete Pile	Pinned	Lateral Deflection, Bending Moment, and Shear Force vs Depth
42-inch Diameter Drilled Pile	Drilled Cast-in-Place Concrete Pile	Pinned	
48-inch Diameter Drilled Pile	Drilled Cast-in-Place Concrete Pile	Pinned	
36-inch Diameter Drilled Pile	Drilled Cast-in-Place Concrete Pile	Fixed	
42-inch Diameter Drilled Pile	Drilled Cast-in-Place Concrete Pile	Fixed	
48-inch Diameter Drilled Pile	Drilled Cast-in-Place Concrete Pile	Fixed	

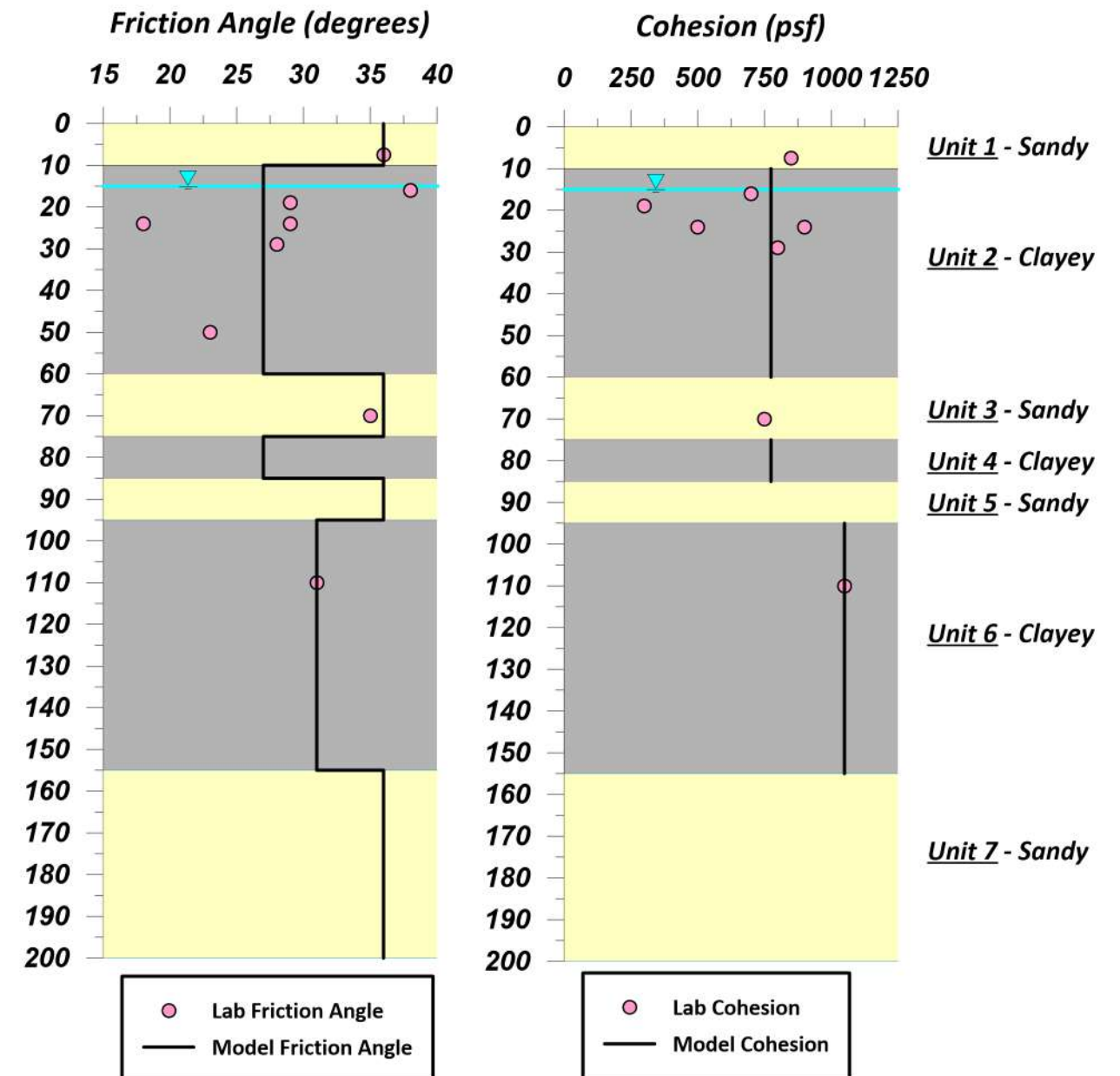
Laboratory Direct Shear Test Results

Borehole ID	Sample No.	Depth (ft)	USCS	Cohesion (psf)	Friction Angle (Degrees)
GP-1	6a	16	SP	700	38
GP-1	9b	29	CL	800	28
GP-2	3a	7.5	CL	850	36
GP-2	9a	24	CL	500	29
GP-3	4b	19	CL	300	29
GP-3	5b	24	CH	900	18
GP-4	10	50	CL	1300	23
GP-4	14	70	SP	750	35
GP-4	22	110	ML	1050	31

Model Soil Parameters

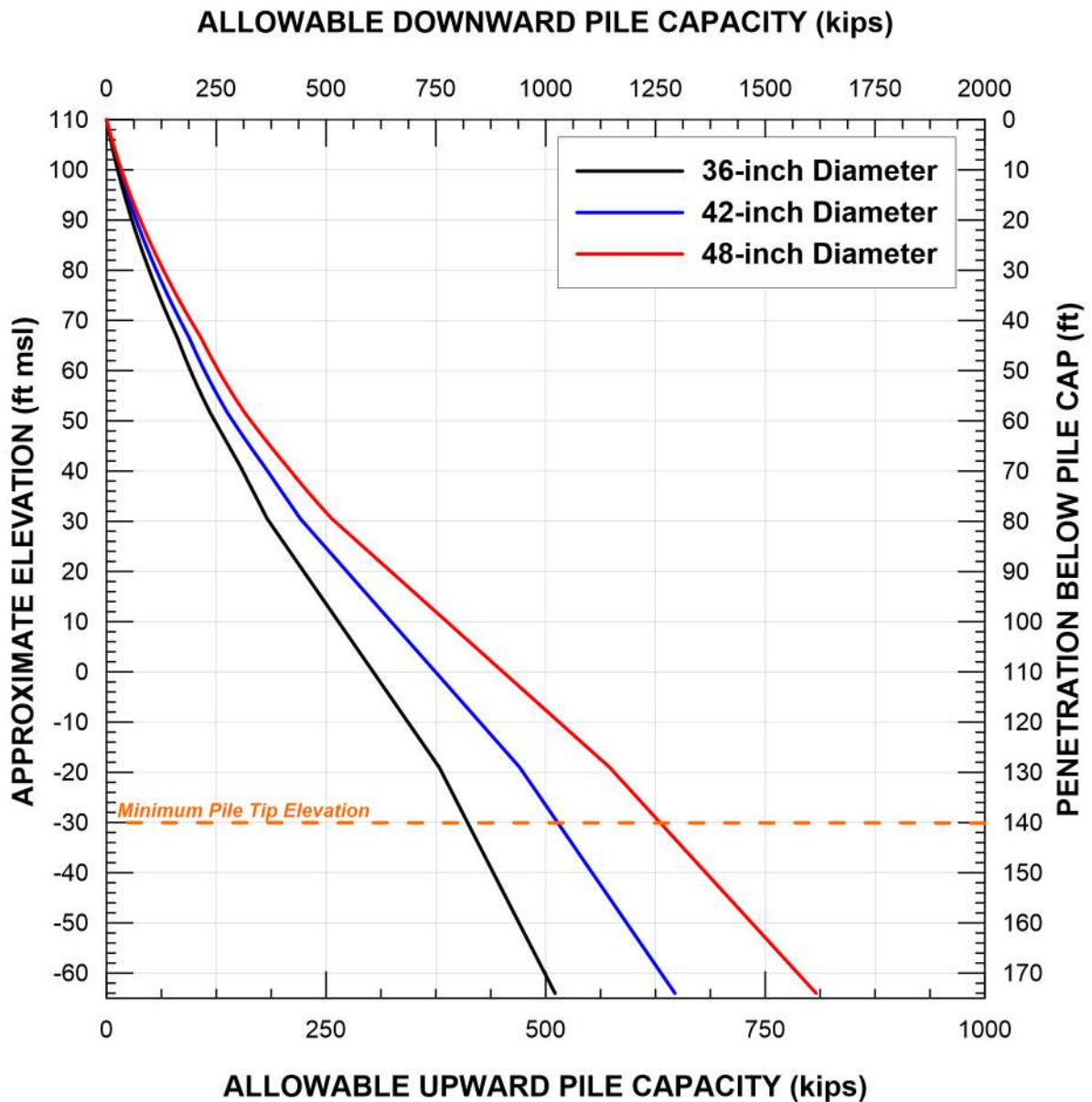
Unit	Depth Range (ft)	Layer Thickness (ft)	Cohesion (psf)	Friction Angle (Degrees)
Unit 1 - Sands	0-10	10	-	36
Unit 2 - Clays	10-60	50	775	27
Unit 3 - Sands	60-75	15	-	36
Unit 4 - Clays	75-85	10	775	27
Unit 5 - Sands	85-95	10	-	36
Unit 6 - Clays	95-155	60	1050	31
Unit 7 - Sands	155-200	45	-	36

Representative Soil Column



Representative Soil Parameters for Pile Analysis

Project: 1056 La Cienega		Figure H-1
Project No.: 21086A	Date: MAR 2022	

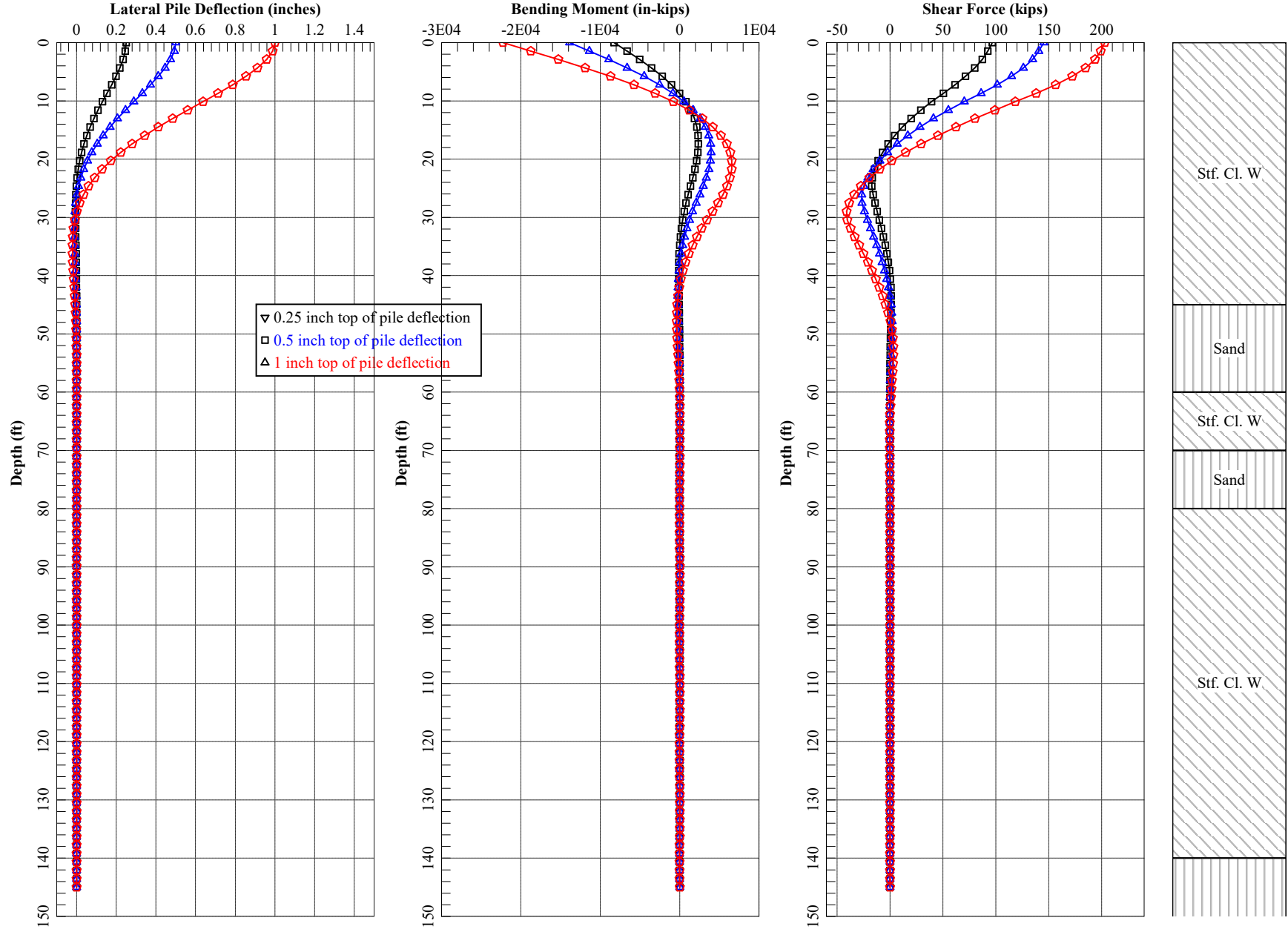


Notes:

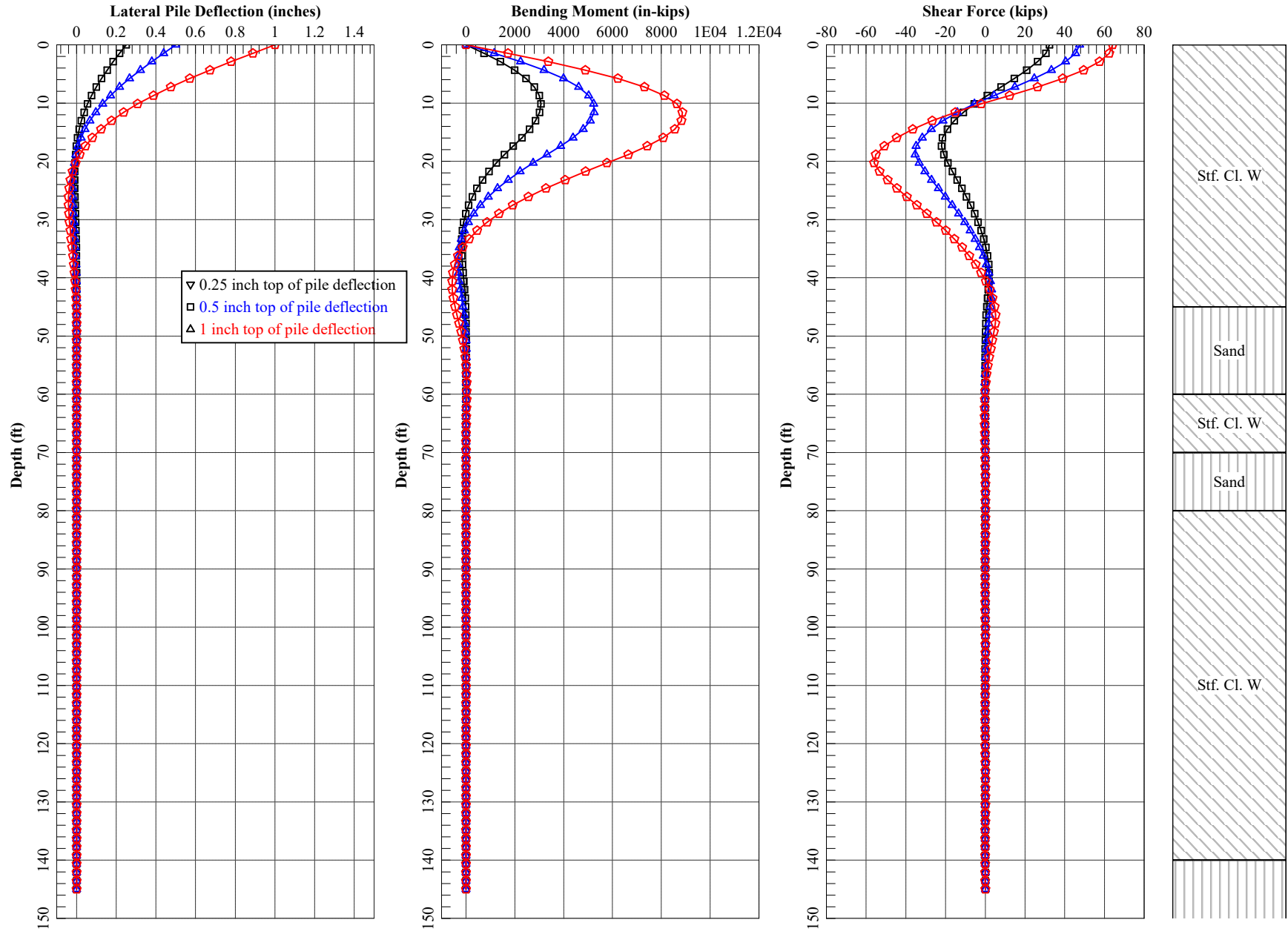
- (1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.
- (2) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.
- (3) Ultimate capacities may be obtained by multiplying the allowable capacities by a factor of 2.
- (4) Piles in groups should be spaced a minimum of 3 pile diameters on centers. Piles should be drilled and filled alternatively with the concrete permitted to set at least 8 hours before drilling an adjacent hole.
- (5) The indicated values assume a Top of Pile depth of 15 ft (from ground surface).

CAST-IN-PLACE DRILLED PILE CAPACITIES

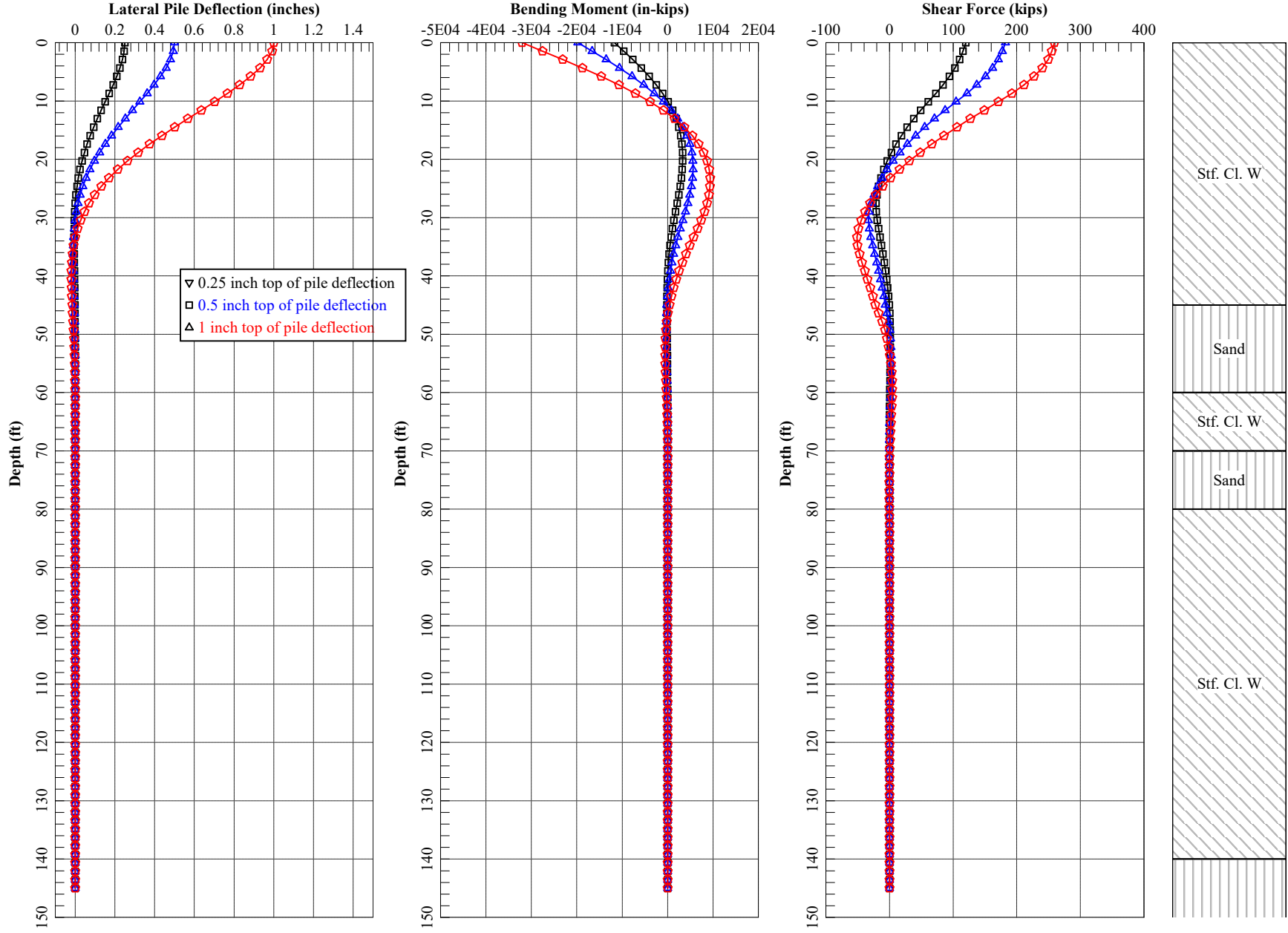
36-inch Diameter Drilled Pile - Fixed Head



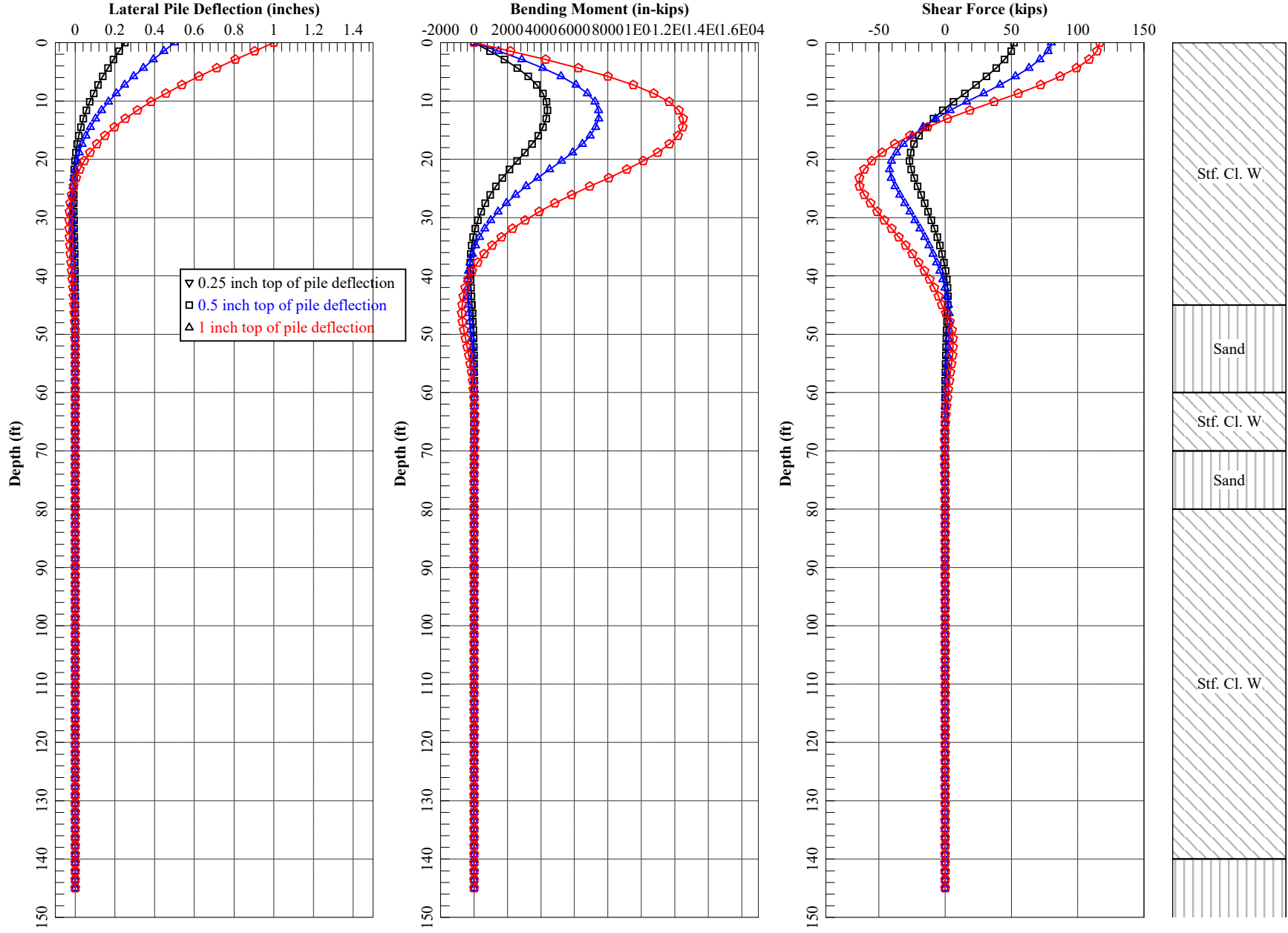
36-inch Diameter Drilled Pile - Pinned Head



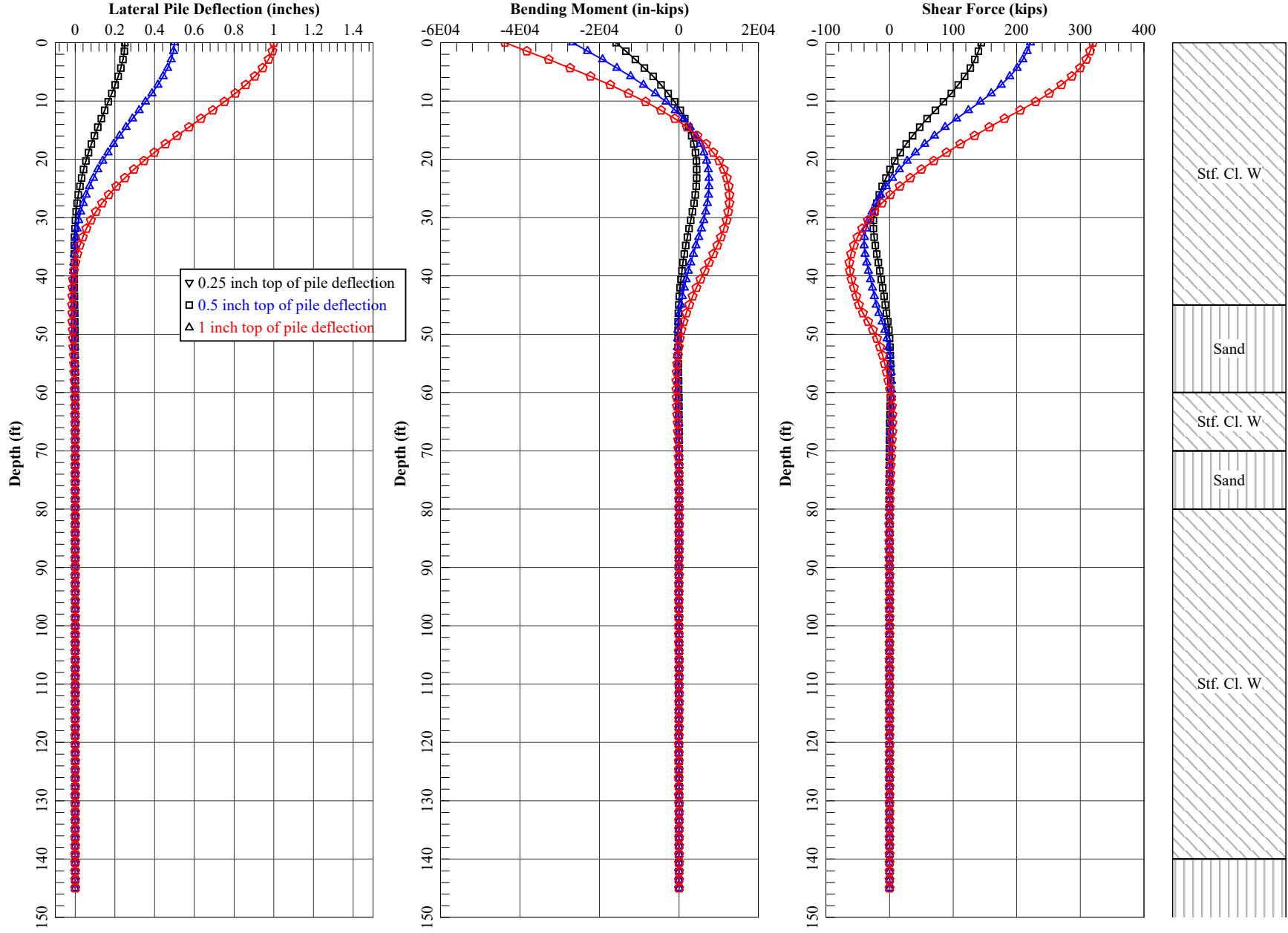
42-inch Diameter Drilled Pile - Fixed Head



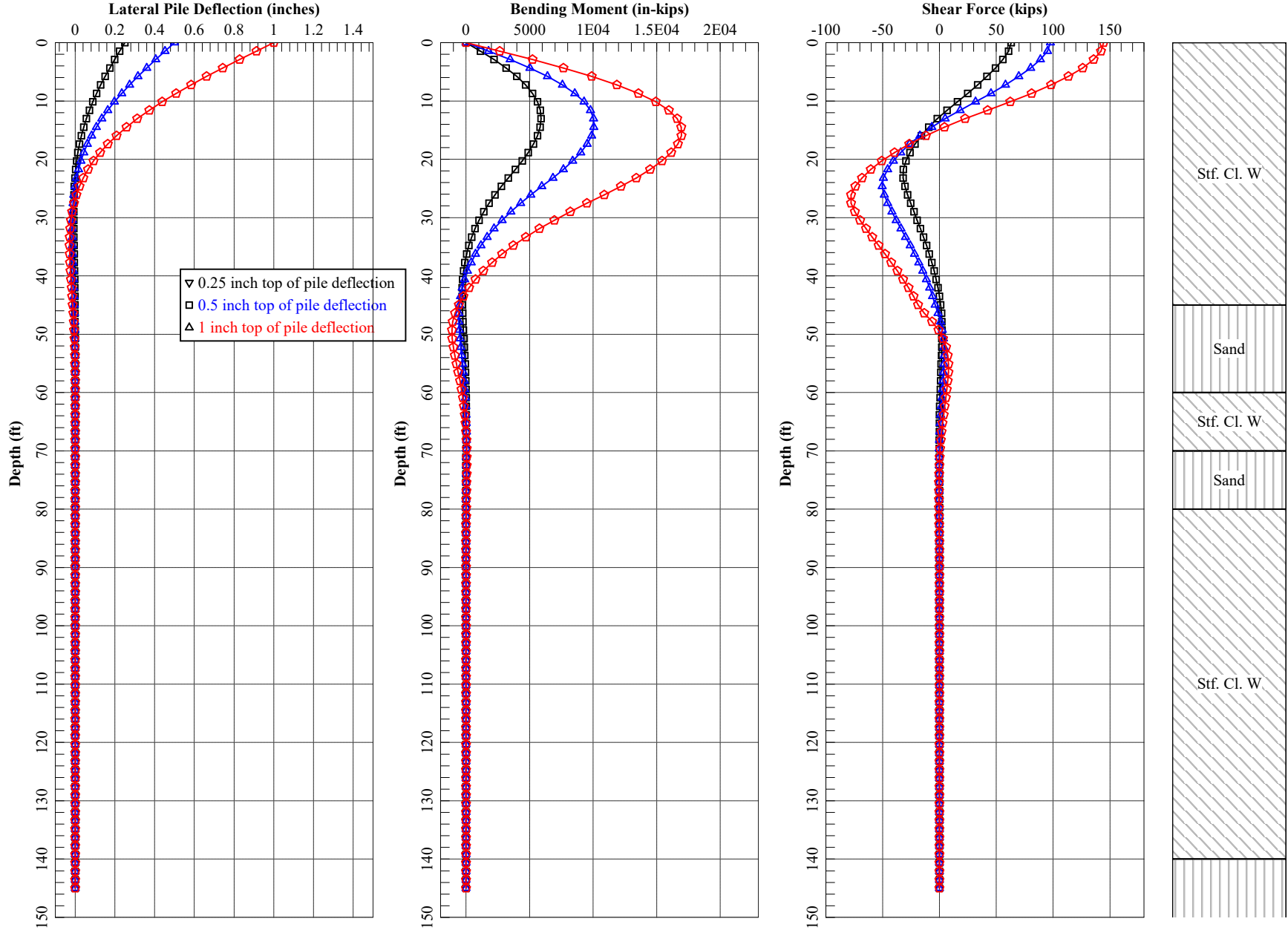
42-inch Diameter Drilled Pile - Pinned Head



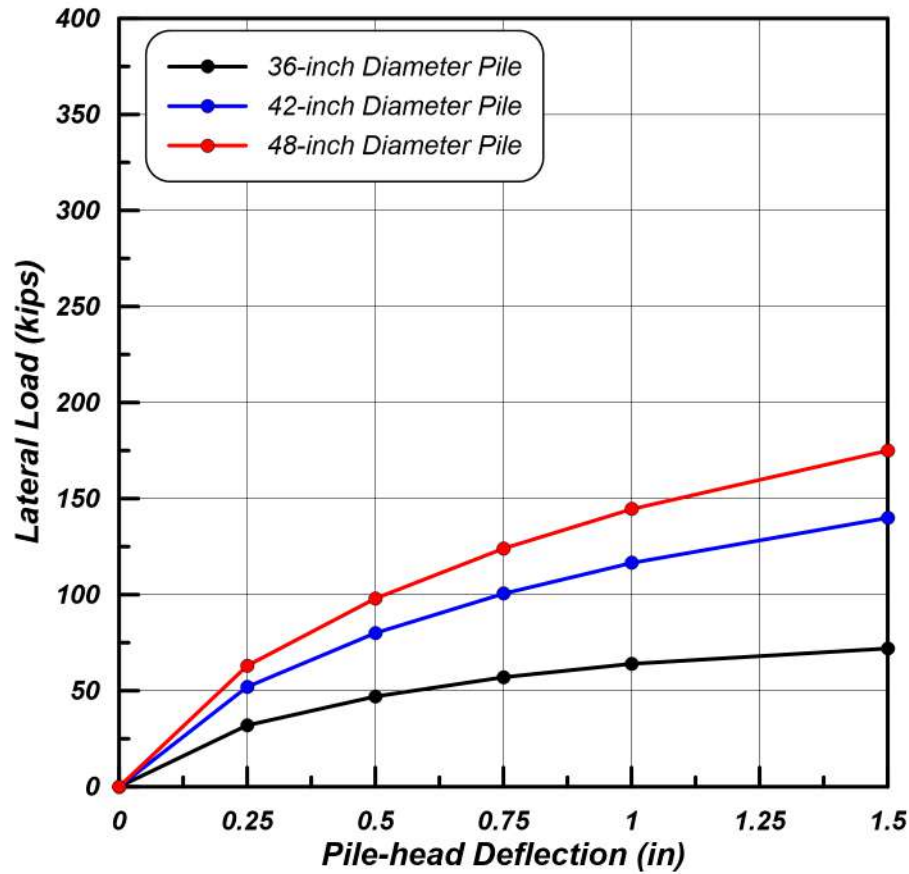
48-inch Diameter Drilled Pile - Fixed Head



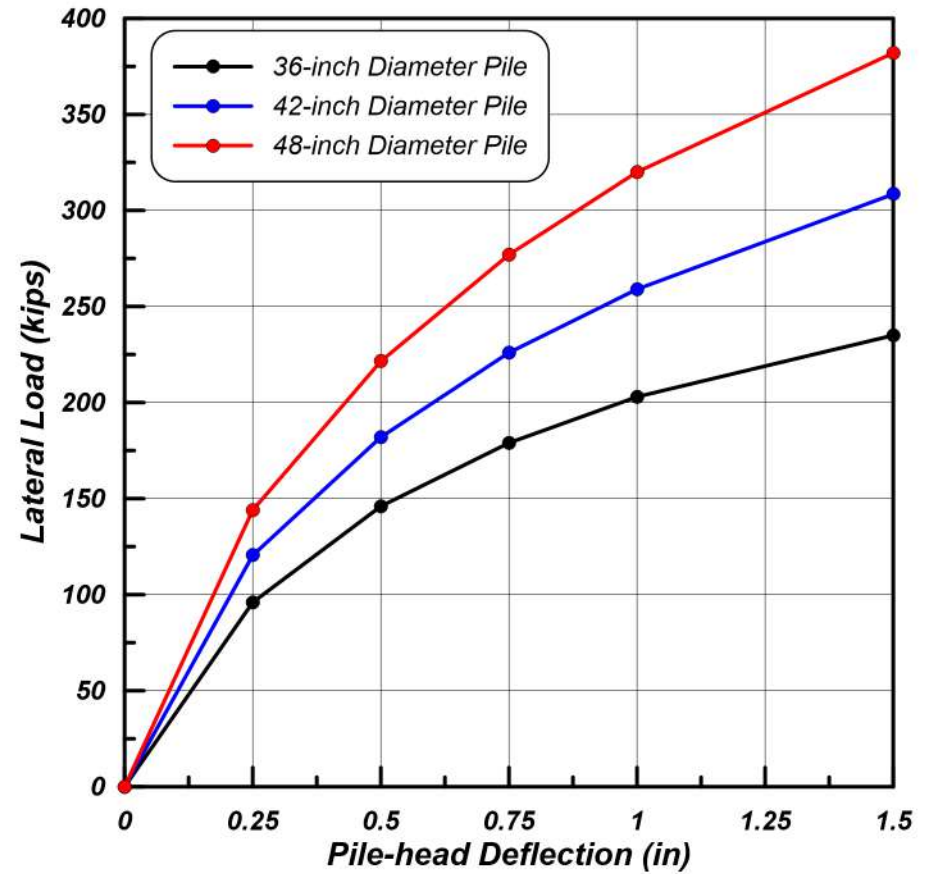
48-inch Diameter Drilled Pile - Pinned Head



Pinned-head Drilled Pile Conditions:



Fixed-head Drilled Pile Conditions:



LATERAL PILE ANALYSIS RESULTS